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Page 78, The discussion by J. W. Richards, Esq., was received subsequent to the publication of the author's closure in *Proceedings*. Dr. Waddell states that he has no comment to make, as he feels sure that Professor Richards is right, notwithstanding Mr. Le Conte's confidence in aluminum steel.

Page 752, line 30, for "$[(kd_1)^2 - y_1^2]^2$" read "$[(kd_1)^2 - y_1^2]$".

Page 753, line 40, for ", Q. The" read ", Q, the".

Page 755, line 36, for "(tan. $\beta + \beta_1$)" read "(tan. β + tan. β_1)".

Page 756, line 8, for "Fig. 27" read "Fig. 29".

Page 759, line 11, for "Fig. 24" read "Fig. 23".

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Page 753, line 10, for "$\_Q$, The" read "Q, the"

Page 755, line 30, for "$\tan\ \theta = \frac{1}{2}$" read "[illegible]"

Page 756, line 8, for "Fig. 20" read "Fig. 20"

Page 756, line 11, for "Fig. 21" read "Fig. 22"

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1313

THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.*

By J. A. L. Waddell, M. Am. Soc. C. E.

With Discussion by Messrs. M. J. Butler, Albert Lucius, Henry W. Hodge, Charles Evan Fowler, L. J. Le Conte, N. Petinot, Leon S. Moisseiff, F. W. Skinner, E. L. Corthell, W. H. Warren, David A. Molitor, Sir Bradford Leslie, John C. Fergusson, V. E. de B. de Broë, M. Séjourné, George L. Norris, J. W. Richards, and J. A. L. Waddell.

Synopsis.

As bridges of exceedingly long span are needed these days in a number of places, it has become necessary to use alloys of high elastic limit and great ultimate strength, in order to lessen materially the dead load to be carried. Up to the present time, nothing more satisfactory or effective than nickel steel has been found; and even that alloy has not been developed to the limit of its practical possibilities. This is due solely to a reluctance on the part of manufacturers of bridge metal to take any special pains in its production.

The time has come to discover some alloy of steel (or some other metal) of great strength, which, in every detail, will be suitable for undergoing the manipulations required of bridge material during the various processes of fabrication.

The objects of this paper are:

First.—To show, for the different span lengths, what the weights of metal per linear foot of bridge would be, in both simple-truss and cantilever structures, when using metals of various elastic limits, ranging from that of ordinary carbon steel up to 100 000 lb. per sq. in.

* Presented at the meeting of April 15th, 1914.

Second.—To indicate the extreme practicable limits of span length for cantilever bridges constructed for the greater part of such materials.

Third.—To demonstrate the comparative economics of finished bridges involved in using metals of the several assumed elastic limits, for all reasonable excess costs of metal per pound, delivered at site, as compared with carbon steel structures.

In the diagrams which form a part of the paper, Figs. 5 and 6 show the weights of metal per linear foot of structure for double-track railway bridges of simple-span and cantilever construction, beginning with short spans and extending to the limits of utility. Directions are given in the text for finding the weights of metal per foot for bridges having any number of tracks and for those designed with other live loads than those adopted in the preparation of the paper. From these diagrams and from the text it will be seen that the possible limits of main span length for cantilever bridges vary from some 2 000 ft. for carbon-steel structures to about 3 500 ft. for those built of metal having an elastic limit of 100 000 lb. per sq. in.

Figs. 10 to 21, inclusive, are diagrams showing the costs of metal erected per linear foot of bridge, in both simple spans and cantilevers, for the different elastic limits assumed. There is a curve for each excess pound price of metal work, delivered at site, above that of carbon steel; and these excesses vary by ½ cent per lb., and extend always slightly beyond the greatest increase that need be considered. On each diagram there is shown, by a line heavier than the others, the cost of erected metal per foot of structure for carbon-steel bridges under the conditions assumed, so as to afford a means of instantaneous comparison of costs between any all-carbon-steel bridge and the corresponding structure built chiefly of the metal to which the diagram pertains.

Figs. 22 and 23 show the comparative economics of all the alloy steels (or other metals) considered, under the assumption that their pound prices for material delivered at the bridge site are the same.

In his paper on "Nickel Steel for Bridges",* the writer showed the results in bridge construction obtainable by the use of nickel steel, and although the limits of attainment indicated therein have not yet

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101.

been reached by American bridge builders, because the metal manufacturers are not willing to guarantee an elastic limit of 60 000 lb. per sq. in. for that alloy, nevertheless, since its publication, a number of important structures have been built of nickel steel, and the manufacturers have indicated to the engineers a willingness to increase their set elastic limit of 50 000 lb., provided they be satisfactorily compensated. The obstacle in the way of adopting a 60 000-lb. elastic limit is the fact that there is no market for the rejected material. The writer is of the opinion, however, that, after a little experience, the manufacturers would have no more trouble in furnishing the alloy with an elastic limit of 60 000 lb. than in furnishing ordinary carbon steel with an elastic limit of 35 000 lb. The best practicable solution of the difficulty is an agreement by the purchaser to pay the manufacturer a fair compensation for his loss by rejections. The writer's reason for thinking that no great trouble will be encountered in manufacturing nickel steel with a minimum elastic limit of 60 000 lb. is that the inspection sheets for the Free Bridge over the Mississippi River at St. Louis, engineered by the late Alfred P. Boller and Henry W. Hodge, Members, Am. Soc. C. E., seldom show an elastic limit materially less than that, although the requirement fixed by arrangement with the manufacturers was only 50 000 lb.

It is the writer's intention, on the first opportunity that presents itself to him for building a bridge of very long span, to construct it of mixed nickel and carbon steels, placing the alloy wherever it can be used to advantage, and arranging, if possible, with the manufacturers for a minimum elastic limit of 60 000 lb., so as to show actually what can be accomplished by adopting nickel steel of that strength, and complying with the specifications given in the before-mentioned paper on "Nickel Steel for Bridges".

A condition which at present militates seriously against the use of nickel steel in bridge building is that the manufacturers ask for it an additional price of some 2 cents per lb., as compared with ordinary carbon steel, although but little more than one-half of that would be a sufficient excess price. This, undoubtedly, is because structural steel manufacturers naturally object to fundamental innovations, preferring to follow lines of least resistance; but, as a number of very long steel spans are contemplated for the near future, they will certainly be forced sooner or later into using high-alloy steels.

Again, it has been stated positively to the writer by Mr. T. L. Willson, the eminent Canadian authority on thermo-chemistry, that ferro-nickel, containing 10% of nickel and nearly 90% of iron, can be furnished with a good profit at 2 cents per lb. for adding to the molten carbon steel in the manufacture of nickel steel, and that no trouble would be involved in burning out all the impurities contained in the said ferro-nickel. This would make the nickel content in the alloy cost about 10 cents per lb. instead of 30 cents, as assumed in the before-mentioned paper. The economics involved by such use of ferro-nickel will be treated hereinafter.

During a recent conversation with two of the largest European producers of nickel, the writer was told that it would be practicable for them to furnish pure nickel in large quantities for the purpose of manufacturing bridge material and to sell this pure nickel at about 20 cents per lb.; but they dropped the hint that they were in no hurry to do so, as the demand for that metal to-day is in excess of the supply, and the main obstacle in the way of large production is likely to be a scarcity of satisfactory nickel ores.

When in France in 1909, the writer's attention was called to the fact that certain metal manufacturers in that country were making, in melts of 5 tons and less, a purified carbon steel, for which rather astonishing claims, in regard to great ultimate strength, high elastic limit, and general suitability for the manufacture of bridges, were made; and on his investigating the matter by both interview and correspondence, he was convinced that these claims might, at least partly, be justified by performance. Thereupon, having some spare time, he prepared an economic study of the possibilities for utilizing such purified steel in bridges, using in his calculations French units, prices, and other conditions, and published the results in French,* under the title, *"Étude Économique de l'Emploi de l'Acier au Carbone à Grande Résistance, pour la Construction des Ponts."*

The excess cost of this purified steel, as compared with ordinary carbon bridge steel, was claimed to be a little less than 1 cent per lb. for the manufactured bridges; and the investigation showed the economics of its use in bridge building for the mean and the extreme conditions of the French metal market, and for a number of elastic limits varying

* *Le Génie Civil*, August 7th, 1909.

from 30 to 45 kg. per sq. mm., the value for French carbon bridge steel being 24 and that for the writer's specified nickel steel 42.5 kg. per sq. mm. The result of the study indicated that there was no advantage for the 30-kg. elastic limit; none for short spans, but a small one for long spans with a 35-kg. elastic limit; a decided saving for all cases with a 40-kg. limit; and a wonderful economy for the 45-kg. limit, the highest elastic limit claimed by any of the French manufacturers.

The writer had hoped that that paper would give an impetus in France (and perhaps elsewhere also) to the manufacture of bridges of purified steel; but, up to the present time, he has not heard of any such development. It is either that the claims of the manufacturers have not been justified by performance, or that the same conditions of inertia and *laisser aller* exist in Europe, in respect to innovations in the manufacture of new bridge metal, as govern in America in relation to the adoption of nickel steel for bridges.

As the future development of America will necessitate the building of many very long-span bridges, it is almost a necessity that there be found an alloy of steel of great strength, high elastic limit, workable under all necessary manipulations in the shops, and of moderate cost. Such an alloy is not going to be discovered by accident, but only by a lengthy and exhaustive series of experiments, laid out systematically in advance, and modified from time to time as knowledge of the subject is accumulated. These experiments should be performed where small melts of steel are readily procurable, and where the experimenter will not be unduly delayed by the metal manufacturers. These conditions are found in France—for there one can procure single ton melts, and possibly smaller ones, if necessary. Arrangements could be made with the mills for rolling into plates and shapes the metal from the various melts without delay; and the experimenter could have at his disposal, not only a first-class testing machine, but also a bridge shop, where all the usual manipulations of the metal might be made. By experimenting at first on different kinds of alloy materials and later on mixtures thereof, one eventually should discover some satisfactory combination that would afford a suitable material of great strength at comparatively moderate cost. The more elaborate and thorough the experiments, the greater the probability of discovering a truly suitable steel of exceedingly high resistance at a reasonable expenditure for production and manufacture.

Such a series of experiments would be expensive, costing fully $100 000, and possibly twice that amount; but the saving in cost on one big bridge alone might easily exceed the entire expenditure. The money required for such experiments could not be obtained from bridge manufacturers, because it is not to their interest to inaugurate changes; but it might be secured from some millionaire philanthropist or from a group of capitalists who contemplate the construction of some great steel bridge.

To show the possibilities for long-span bridge construction by the use of such superior alloys of steel, and the economics thereof, is the object of this paper.

The basis of the following investigation is a mass of diagrammed and tabulated data concerning weights of metal in simple-span and cantilever bridges of carbon steel, up to a limit of 600-ft. spans for the former and 1 800-ft. main openings for the latter, accumulated by the writer and his firm during the last quarter of a century, and the weights of nickel-steel bridges and of mixed nickel-steel and carbon-steel bridges computed by the writer in the preparation of his before-mentioned paper on "Nickel Steel for Bridges". Because the said diagrammed weights of metal per linear foot of span in simple-truss bridges are limited to lengths of 600 ft. it has been found necessary, in preparing this paper, to extend them to 1 000 ft. by making actual calculations of stresses, sections, and weights of metal for several long spans, using the various kinds of steel assumed. From these recorded and specially computed weights, and by the succeeding formulas of reduction and extension, have been prepared the various diagrams of weights of metal per linear foot of span contained in this paper.

Many preliminary and tentative diagrams of curves were constructed, but, for considerations of space and cost, they are not included herein. These individual diagrams, prepared for each class of steel and each type of bridge, divided the weights into those for "Floor System"; "Lateral System"; "Trusses"; and "On Piers"; but in the selected diagrams only the total weights of metal per linear foot of span for a combination of all the parts of the structures are recorded, as these quantities constitute the results sought, and to be used in the succeeding economic investigation. Near the end of this paper, however, are given some tables from which can be found the approximate values of weights of metal per linear foot of span for "Floor

Systems"; "Lateral Systems"; and "On Piers"; and from these and the diagrams, Figs. 5 and 6, can be computed the weights for "Trusses".

The weights given for bridges of carbon steel are based on the standard specifications for designing contained in the writer's "De Pontibus". They are quite accurate up to the before-mentioned limits of 1 000 ft. for simple spans and 1 800 ft. for the main openings of cantilever bridges. Beyond these limits (shown on the diagrams by dotted lines) they are to a certain extent conjectural, although, in all probability, not far from correct, because of the thorough consideration which has been given to all factors influencing their values.

The weights for steel having an elastic limit of 60 000 lb. per sq. in. are those obtained for the diagrams of the paper on "Nickel Steel for Bridges", and are based on the specifications contained in that paper. They are quite accurate up to the before-mentioned limits of 1 000 ft. for simple-truss spans and 1 800 ft. for the main openings of cantilever bridges; and a comparison of the curves for cantilevers of openings from 1 200 to 2 600 ft., shown in Fig. 71 of the writer's Nickel Steel paper with those of Fig. 6 of this paper indicates, in general, an excellent agreement. In the former the curves beyond 1 800-ft. openings were determined by an assumed regularity of continuity, as explained in that paper, and in the latter they were computed by the use of Equation 19 of this paper.

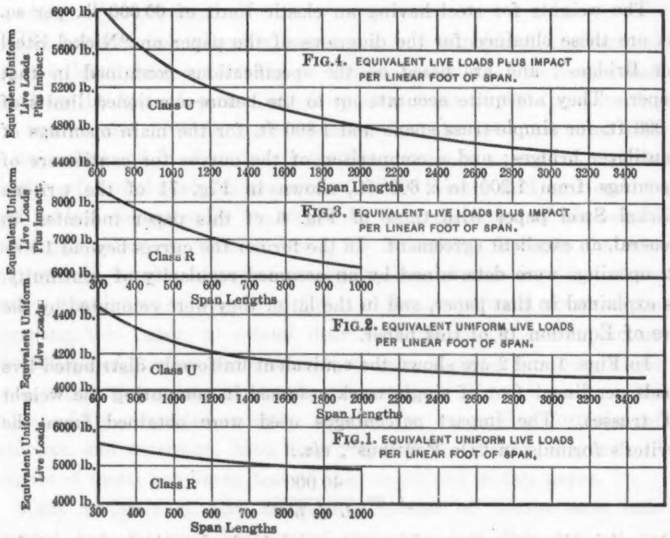
In Figs. 1 and 2 are shown the equivalent uniformly distributed live loads per linear foot of single track assumed in computing the weight of trusses. The impact percentages used were obtained from the writer's formula in "De Pontibus", *viz.:*

$$I = \frac{40\,000}{L + 500},$$

where I is the percentage of impact and L is the length, in feet, of the portion of the bridge which must be covered by the moving load in order to obtain the maximum stress on the member under consideration. Although it is true that the writer has evolved a more accurate formula than this, based on some lately made experiments on impact in railway bridges from moving loads, he has decided to adhere to the old formula so as not to necessitate the changing of the weight curves on which this investigation is primarily founded. Figs. 3 and 4 give a combination of the equivalent uniform live loads and the impacts therefrom. The loads obtained from these curves added to the dead loads

make the total loads per linear foot on the bridges, from which total loads the load ratios of reduction are found.

The following are the formulas of reduction used in passing from known weights of metal per linear foot of span in carbon-steel bridges to the corresponding weights in alloy-steel bridges. An observation of the nomenclature will show that the unaccented capital letters severally represent weights of metal per linear foot of span in carbon-steel bridges (or otherwise known weights of bridges of any kind of steel), and the accented capital letters, the corresponding weights for alloy-steel bridges (or otherwise the corresponding unknown weights of



bridges of some other kind of steel), also that the small letters severally represent linear dimensions of structures, the main exceptions being that capital R is used for reactions and small r for ratios.

Floor System.—

Let F = weight of metal per linear foot of span in the "Floor System" of carbon-steel bridges.

L = ditto for "Lateral System".

T = ditto for "Trusses".

P = ditto for "On Piers", including anchorage material in the case of cantilever bridges.

A certain portion of the weight of the floor system will vary inversely with the elastic limit of the steel, and the remainder will be invariable.

Let V = the variable portion,

and I = the invariable portion.

$$\text{Then } F = V + I \quad \ldots\ldots (1)$$

Let F' = the weight of metal per linear foot of span in the floor system of alloy-steel bridges,

and r (greater than unity) = the ratio of elastic limits of alloy steel and carbon steel.

$$\text{Then } F' = I + \frac{V}{r} \quad \ldots\ldots (2)$$

In heavy, double-track bridges, and especially those of long span, I will be approximately 0.35 F, and V approximately 0.65 F, hence

$$F' = 0.35\,F + \frac{0.65\,F}{r} = F\left(0.35 + \frac{0.65}{r}\right) \quad \ldots\ldots (3)$$

In dealing with spans of greater length than any of those yet actually computed, it must not be forgotten that the increasing width of structure will augment the weight of the floor-beams and, consequently, the weight of metal per linear foot of span for the floor system. In case of double-track bridges, an economy can be effected by widening the cantilever arms and the anchor arms uniformly from ends to supporting pier; but it is probable that motives of policy would lead the projectors to construct exceedingly long spans so as to carry more than two tracks.

Lateral System.—

Let l_1 = length of span at which it pays to begin to use high steel for the laterals beyond the ends of l_1, it being assumed that the weight of laterals is uniform over the entire length, l_1, or, in other words, that minimum sections are used therein throughout;

R_1 = wind reaction at end of l_1;

R = wind reaction at end of span l;

r_w = ratio (greater than unity) of R and R_1;

L_1 = weight of carbon steel per linear foot for lateral system over the length, l_1;

L'_2 = weight of mixed carbon and alloy steels per linear foot of span at end of span, l:

$$\text{Then } L'_2 = L_1 \left(0.3 + 0.7 \frac{r_w}{r}\right).$$

Let L'_a = average weight of metal per linear foot for entire span, l,

$$\text{Then } L'_a = \frac{1}{l}\left\{L_1 l_1 + \frac{L'_2 + L_1}{2}(l - l_1)\right\} \quad \text{.........(4)}$$

Should L'_2 figure less than L_1, it shows that near the ends of the span minimum sections of the high steel must be used and that L'_a will equal L_1.

In passing beyond the limits of actually figured spans, when computing the weights of metal in lateral systems, it must be remembered that, as just explained for the floor system, the weight per foot is increased, not only because of the greater span length but also because of the greater span width. As a rule, it may be stated that, for any very long span (the length thereof remaining constant), the effect of increasing the width between central planes of trusses $n\%$ is to increase the weight of metal in the lateral system about $\frac{n}{2}$ per cent.

Trusses.—In respect to the weight, T, of metal per linear foot of span for trusses of carbon steel, the following equation may be used:

$$T = K + T_1 + C_c + C_w \quad \text{................(5)}$$

where K is the portion of the total truss weight per linear foot which is independent of the quality of the metal and of the stresses; T_1 is that of the main portions of the tension members and of their details that are directly affected by the stresses; C_c is that of the main portions of the compression chords and inclined end posts and their details that are directly affected by the stresses; and C_w is that of the main portions of the compression web members.

From experience in designing large bridges it may be stated that, as an average,

$$K = 0.2T$$

$$T_1 = 0.3T$$

$$C_c = 0.3T$$

$$C_w = 0.2T$$

Both T_1 and C_c (and consequently their sum) will vary inversely with the elastic limit of the metal; but C_w, on account of the influence of the ratio of strut length to least radius of gyration, will not vary in that ratio. As an approximation it may be assumed that, in passing from any grade of steel to a higher grade, if, as before, r (greater than unity) is the ratio of the elastic limits of the two metals,

$$C'_w = \frac{1}{2}\, C_w\left(1 + \frac{1}{r}\right) \qquad (6)$$

$$\text{and } C'_c = \frac{C_c}{r} \qquad (7)$$

Substituting these values in Equation 5, we have

$$T' = K + \frac{1}{r}\left(T_1 + C_c\right) + \frac{1}{2}\, C_w\left(1 + \frac{1}{r}\right) \qquad (8)$$

Substituting the values of K, T_1, C_c, and C_w in terms of T as previously given, we have

$$T' = T\left(0.3 + \frac{0.7}{r}\right) \qquad (9)$$

In finding the new truss weight per linear foot for a higher steel, after computing it (as just indicated) for the direct effect of increased elastic limit, it must be corrected for the indirect effect, which is the changed total load per linear foot for trusses. This correction is made thus:

Find the sum of the live load, impact load, and dead load per linear foot of span, for the known truss weight, T, and then determine approximately the corresponding sum (on the basis of an assumed final value of T'_f) for the new truss weight. Let the ratio of these sums (less than unity) be r_1, then

$$T'_f = T'(0.3 + 0.7\, r_1) \qquad (10)$$

where T'_f is the final value of the truss weight. Combining Equations 9 and 10 gives

$$T'_f = T\left(0.3 + \frac{0.7}{r}\right)(0.3 + 0.7\, r_1) \qquad (11)$$

If the computed value of T'_f does not agree quite closely with its value adopted in determining the trial dead load, a new dead load is to be assumed, and the calculations are to be made afresh. The second attempt, in all probability, will give a sufficiently accurate agreement.

On Piers.—To find the new value, P', from the old value of P, the span length being unchanged, the following approximately correct equation may be used:

$$P' = P\left(0.6 + 0.4\,\frac{r_1}{r}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where r and r_1, respectively, are the ratios previously indicated for elastic limits and total loads per linear foot of span.

In extending a curve of simple truss weights of metal per linear foot of span beyond the limits of accurate computations, the following formulas may either be used directly or as a check, the character of the steel, of course, being unchanged. Assume first that the live and the dead loads per linear foot of span remain constant, and consider the effect only of longer spans and greater truss depths. Dealing first with the chords, some 85% of their weights of metal per linear foot of span vary directly as the moments of the total loads and inversely as the truss depths; but the moments vary as the squares of the span lengths, and the stresses are inversely as the truss depths. Again, the truss depths within short limits may, without serious error, be taken to vary directly as the span lengths. Such being the case, 85% of the weights per foot of the chords will vary directly as the span lengths, or

$$C' = 0.15\,C + 0.85\,C\,\frac{l'}{l} = C\left(0.15 + 0.85\,\frac{l'}{l}\right) \ldots (13)$$

where C is the chord weight per foot for the shorter span, l, and C' is the corresponding weight for the longer span, l'.

Let W and W' be, respectively, the weights of metal per linear foot of span in the webs of the two spans. About 75% of these will vary directly as the averages of all the live-load and dead-load shears on the spans, and these average shears vary almost directly as the span lengths. Again, the said 75% of W and W' will vary directly as the truss depths, and, therefore, as previously assumed, once more directly as the span lengths.

Combining these ratios will give the equation:

$$W' = 0.25\,W + 0.75\,W\left(\frac{l'}{l}\right)^2 = W\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\} \ldots (14)$$

but $T = C + W$,

$$\text{and } T' = C' + W' = C\left(0.15 + 0.85\,\frac{l'}{l}\right) + W\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\} .(15)$$

It is well known that in trusses with parallel chords and of economic depths the weight of the chords is equal to the weight of the web; but, in trusses with polygonal chords and having center depths less than the theoretically economic ones, as do those of all long-span bridges, the weight of the chords is much greater than that of the web. As a general average, we may assume that $C = 0.6\ T$, and $W = 0.4\ T$, hence

$$T' = 0.6\ T\left(0.15 + 0.85\frac{l'}{l}\right) + 0.4\ T\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\}$$

$$= T\left\{0.19 + 0.51\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\} \ldots\ldots\ldots\ldots (16)$$

This value of T' is based on the incorrect assumption that the total loads per linear foot of span are the same for both spans under consideration, hence it requires a further modification, as follows:

$$T'_f = T'(0.2 + 0.8\ r_1) \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

where T'_f is the final value of the weight of truss metal per linear foot of the longer span, and r_1 (in this case greater than unity) is the ratio of the total loads per linear foot.

Combining Equations 16 and 17, we have

$$T'_f = T\left\{0.19 + 0.51\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\}(0.2 + 0.8\ r_1) \ldots (18)$$

A test of this formula, on carefully computed curves of truss weights for simple spans of nickel steel from 600 to 1 000 ft. in length, shows that slightly undue prominence has been given to the invariable portion of the weights, and that the following modification of the formula will give more accurate results:

$$T'_f = T\left\{0.15 + 0.55\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\}(0.15 + 0.85\ r_1) \ldots (19)$$

This last formula, when tested on the truss weights of simple spans from 700 to 1 000 ft. in length for an elastic limit of 90 000 lb., gave exceedingly close results; hence it is proper to adopt it as the equation for extension of all truss weights for simple spans, and, inferentially, for those of cantilever bridges; in fact, it has been tested on some of the actually computed truss weights of cantilever bridges and found to give excellent agreement.

Attention is called to the semi-rational, semi-empirical character of these reduction and extension formulas. They are, in general, the result of long personal experience in the quick computation of metal

weights for bridges; but they have been modified slightly, as hereinbefore indicated, to agree with certain checks that have been made in this investigation. As far as practicable, the formulas of Equations 11 and 19 were used for checking each other; and the results of such checks were always satisfactory. For instance, if a curve of truss weights for one class of steel were used as a basis for finding, by Equation 11, the corresponding curve for another class of steel, the latter curve would be checked by starting from any desired point (generally where the weights of actually computed bridges cease) and passing, by using Equation 19, from one span length to another, 100 or 200 ft. greater, and continuing in this manner to the superior end of the curve.

Fig. 5 gives the total weights of metal per linear foot of span in simple-truss bridges with Class R live load. A glance at it shows the great saving in weight of metal obtained by using any of the alloy steels in preference to carbon steel. The difference is most apparent between the weights for $E = 50\,000$ lb. (the nickel steel that the metal manufacturers are willing to furnish) and those for $E = 60\,000$ lb. (the nickel steel used by the writer in his series of experiments some 8 or 10 years ago, and which alloy could be readily produced by the manufacturers through the expenditure of a little extra care and without any greater expense worth mentioning). The gradual reduction in the saving of metal with the increase of elastic limit is strikingly noticeable; and the conclusion may be drawn that, unless the extremely high-alloy steels can be obtained with only moderate increase in cost, there will be no economy in using them in simple-span bridges.

Fig. 6 gives the average total weights of metal per linear foot of span for cantilever structures having main openings of various lengths. The type of these bridges is the most common one (denominated "Type A" in "Nickel Steel for Bridges"), the live loads used being Classes R and S for the floor system and Class U for the trusses, as given in "De Pontibus." The proportional dimensions of this typical, through, cantilever bridge are as follows:

A main span, l, having a suspended span of three-eighths of l, and two cantilever arms each five-sixteenths of l in length, also two anchor arms of the same length as the cantilever arms. Any reasonable variation from these proportions would not change materially the average weight of metal per linear foot of span, as given by the curves

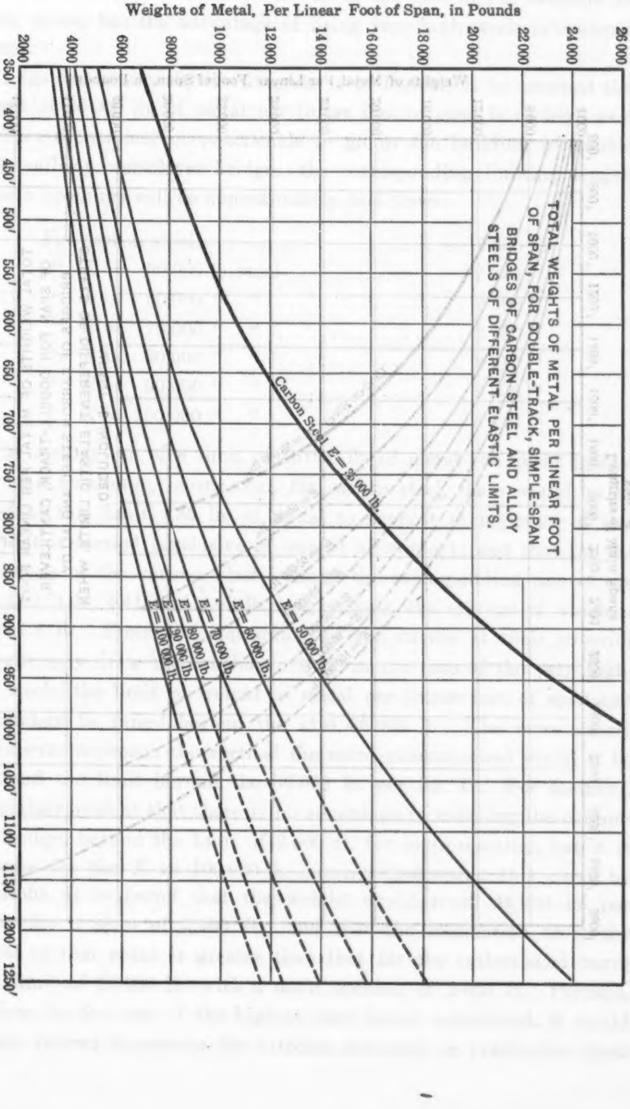

Weights of Metal, Per Linear Foot of Span, in Pounds
2000
4000
6000
8000
10 000
12 000
14 000
16 000
18 000
20 000
22 000
24 000
26 000
TOTAL WEIGHTS OF METAL PER LINEAR FOOT OF SPAN, FOR DOUBLE-TRACK, SIMPLE-SPAN BRIDGES OF CARBON STEEL AND ALLOY STEELS OF DIFFERENT ELASTIC LIMITS.
Carbon Steel E = 35 000 lb.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

Fig. 5.

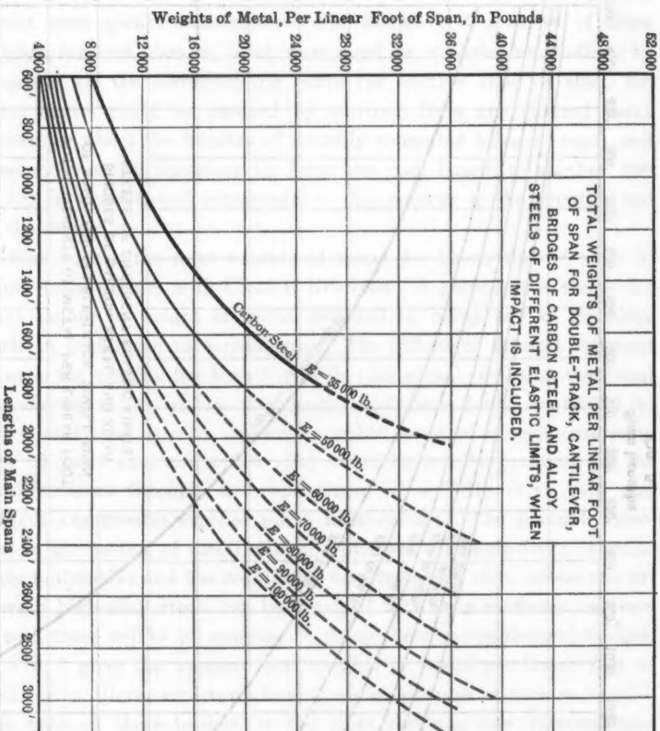

TOTAL WEIGHTS OF METAL PER LINEAR FOOT OF SPAN FOR DOUBLE-TRACK, CANTILEVER, BRIDGES OF CARBON STEEL AND ALLOY STEELS OF DIFFERENT ELASTIC LIMITS, WHEN IMPACT IS INCLUDED.
Weights of Metal, Per Linear Foot of Span, in Pounds
4000
8000
12000
16000
20000
24000
28000
32000
36000
40000
44000
48000
52000
Lengths of Main Spans
600'
800'
1000'
1200'
1400'
1600'
1800'
2000'
2200'
2400'
2600'
2800'
3000'
3200'
3400'
3600'
Carbon Steel
E = 35000 lb.
E = 50000 lb.
E = 60000 lb.
E = 70000 lb.
E = 80000 lb.
E = 90000 lb.
E = 100000 lb.

Fig. 6.

on Fig. 6. In this figure the superiority of the alloy steels over the carbon steel is just as clear as it was in the case of the diagram for simple spans, but the advantage of using very high steels is evidently greater.

If (as can be seen by the diagram to be logical) it be assumed that a limit of 36 000 lb. of metal per linear foot of span is as high as it is either economical or practicable to go in the building of double-track, railway, cantilever bridges, the corresponding limiting lengths of main openings will be approximately as follows:

For carbon steel	2 030 ft.
" $E=$ 50 000-lb. steel	2 340 "
" $E=$ 60 000 " "	2 590 "
" $E=$ 70 000 " "	2 780 "
" $E=$ 80 000 " "	2 910 "
" $E=$ 90 000 " "	3 030 "
" $E=$ 100 000 " "	3 140 "

The assumption of a limit of 36 000 lb. of metal per linear foot of span as a maximum, means that, for carbon steel, there would be required at this limit 4.35 lb. of metal to support each pound of live load that is carried (exclusive of impact allowance); and that for the alloy steels of the various elastic limits the corresponding figures are 4.37, 4.39, 4.40, 4.41, and 4.42 lb., respectively, the average of which is about 4.4 lb. From the appearance of the curves at their superior ends one may draw the conclusion that, in the case of the very high-alloy steels, the limit of weight of metal per linear foot of span can legitimately be raised beyond the said 36 000 lb. The more nearly these curves approach the vertical the more uneconomical would it be to extend the limit beyond the 36 000 lb. per lin. ft. For instance, it is plainly evident that there is no advantage in carrying the carbon-steel bridges beyond the limit of 2 000 ft. for main opening, but it is otherwise for the $E=$ 100 000-lb. curve. Continuing this curve by deflections, it is found that the weight would reach 46 000 lb. per lin. ft. for a span of 3 400 ft.; and that the inclination from the vertical at that point is greater than that for the carbon-steel curve at its limit of 36 000 lb. with a main opening of 2 030 ft. Perhaps, therefore, in the case of the highest steel herein considered, it would be more correct to assume the extreme economic or practicable limit

of main opening to be 3 400 ft. or even 3 500 ft. For this last length the average weight of metal per linear foot of structure shown by the $E = 100\,000$-lb. curve would be 52 000 lb., which means that it would require 6.38 lb. of metal to support each pound of live load, exclusive of the effect of impact. This seems to be an excessive quantity; nevertheless, it is conceivable that conditions might exist which would render it advisable to adopt this extreme limit of main opening, although, at such a length, a suspension bridge is undoubtedly cheaper than a cantilever structure.

If it be admitted that—as is maintained by some bridge engineers —in structures of very long span the impact of the live load on the main members of trusses is essentially nil, the practicable limiting length of main opening will be somewhat increased. Moreover, such a contention is not far wrong, because the latest experiments on impact from live loads on bridges show that the effect thereof on ordinarily long spans is much less than engineers in general have been assuming during the last two decades. That the impact ever reduces actually to zero is most unlikely; but, for openings of 1 200 ft. and greater, it is true that its amount is so small as to be negligible, in view of the fact that the live-load stresses on the main truss members will never be quite as great as they are computed, because, first, the trains on the two tracks never advance together so as to produce maximum web stresses; second, such trains are not likely ever to cover entirely the bridge or even any individual part of it, except, perhaps, the central span; and, third, it is improbable that any load of cars, unless they be ore or coal cars, will ever be uniformly full and loaded to the assumed limit.

In Fig. 7 are given the curves of weights for cantilever bridges of the same type and loading as those used in preparing the curves in Fig. 6, excepting that the impact on main members of trusses is assumed to be equal to zero. The curves in Fig. 7 begin at main openings of 1 200 ft. and extend to the greatest practicable limiting lengths of such openings. A comparison of Figs. 6 and 7 shows that, by ignoring impact on trusses, there is, on the average, a saving of some 700 lb. of metal per linear foot of span, for all spans and all kinds of steel. This difference, curiously enough, is comparatively uniform for all the curves and over their entire lengths, with a few exceptions. At first thought, one might imagine that the saving should be greater

for long spans than for short ones, but it must be remembered that, as the span length increases, the percentage of live-load impact diminishes. It is this fact which makes the difference under consideration so uniform.

Under the assumption that the limit of weight of metal is 36 000 lb. per lin. ft., the greatest practicable span lengths have been

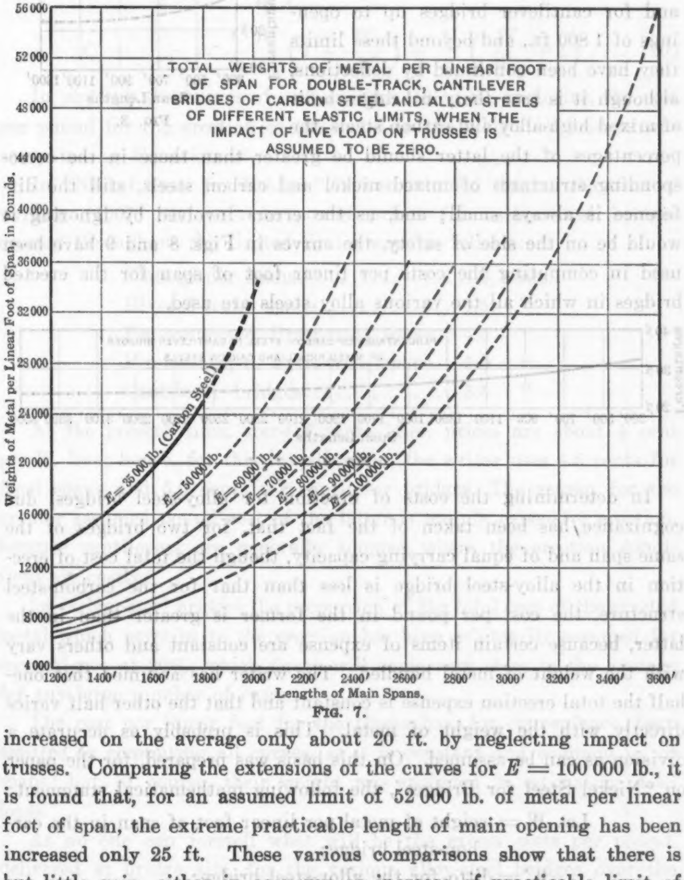


FIG. 7.

increased on the average only about 20 ft. by neglecting impact on trusses. Comparing the extensions of the curves for $E = 100\,000$ lb., it is found that, for an assumed limit of 52 000 lb. of metal per linear foot of span, the extreme practicable length of main opening has been increased only 25 ft. These various comparisons show that there is but little gain, either in economy or increase of practicable limit of opening, by neglecting the effect of impact; hence, in the economic

investigations which follow, the effect of impact has been duly considered.

In Figs. 8 and 9 are plotted, for both simple spans and cantilever bridges, the percentages of carbon steel in structures of mixed nickel and carbon steels. The curves are accurate for simple spans up to 600 ft. and for cantilever bridges up to openings of 1 800 ft., and beyond these limits they have been continued by deflections, although it is true that, in bridges built of mixed high-alloy and carbon steels, the percentages of the latter should be greater than those in the corresponding structures of mixed nickel and carbon steels, still the difference is always small; and, as the errors involved by ignoring it would be on the side of safety, the curves in Figs. 8 and 9 have been used in computing the costs per linear foot of span for the erected bridges in which all the various alloy steels are used.

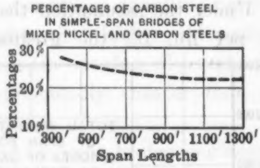


FIG. 8.

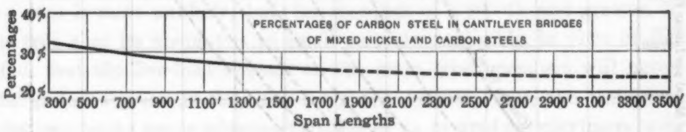


FIG. 9.

In determining the costs of erection for alloy-steel bridges, due cognizance has been taken of the fact that, for two bridges of the same span and of equal carrying capacity, though the total cost of erection in the alloy-steel bridge is less than that for the carbon-steel structure, the cost per pound in the former is greater than in the latter, because certain items of expense are constant and others vary with the weight of metal handled. The writer has assumed that one-half the total erection expense is constant and that the other half varies directly with the weight of metal. This is probably as accurate a division as can be assumed. On this basis was prepared, for the paper on "Nickel Steel for Bridges", the following mathematical statement:

Let W = weight of metal per linear foot of span in the carbon-steel bridge;

W' = ditto for the alloy-steel bridge;

C = cost per pound for erecting the carbon-steel bridge;

C' = cost per pound for erecting the alloy-steel bridge;

F' = cost per linear foot for erecting the alloy-steel bridge;

then $C\ W$ = cost per linear foot for erecting the carbon-steel bridge.

$$F' = \frac{C\ W}{2}\left(1 + \frac{W'}{W}\right),$$

$$C' = \frac{F'}{W'} = \frac{C\ W}{2\ W'}\left(1 + \frac{W'}{W}\right) = \frac{C}{2}\left(\frac{W}{W'} + 1\right) \ldots\ldots (20)$$

In plotting the curves of cost in Figs. 10 to 21, inclusive, the cost per pound for the erection of the metal in the alloy-steel bridges was computed by Equation 20.

In "Nickel Steel for Bridges" it was assumed that at the time of writing (1907) the average pound prices for carbon-steel bridges erected throughout the United States were as follows:

Plate-girder spans..............	4.0	cents.
Riveted-truss spans............	4.5	"
Pin-connected, Pratt-truss spans..	4.5	"
Pin-connected, Petit-truss spans..	5.0	"
Cantilever bridges.............	5.5	"

At the present time, some 6 years later, prices are about ½ cent per lb. less; hence, for this investigation, the writer uses 4.5 cents for fixed spans and 5.0 cents for cantilever bridges. The reason for the greater assumed pound cost of the latter is mainly expensive erection, because cantilevers are generally adopted where the erection conditions are costly.

In carbon-steel bridges the proportion of the total cost of the erected metal which pertains to the erection has been arbitrarily assumed for convenience at 20%, which figure will come very close to the average for any large number of cases.

The cost per linear foot for the tracks has, for convenience, been omitted in computing all curves, as it is constant and common to all spans; but, of course, their weight was included in estimating dead loads.

As no one can foretell what will be the excess costs per pound, delivered at bridge site, for the various alloy-steel bridges over the corresponding costs for carbon-steel bridges, in this investigation

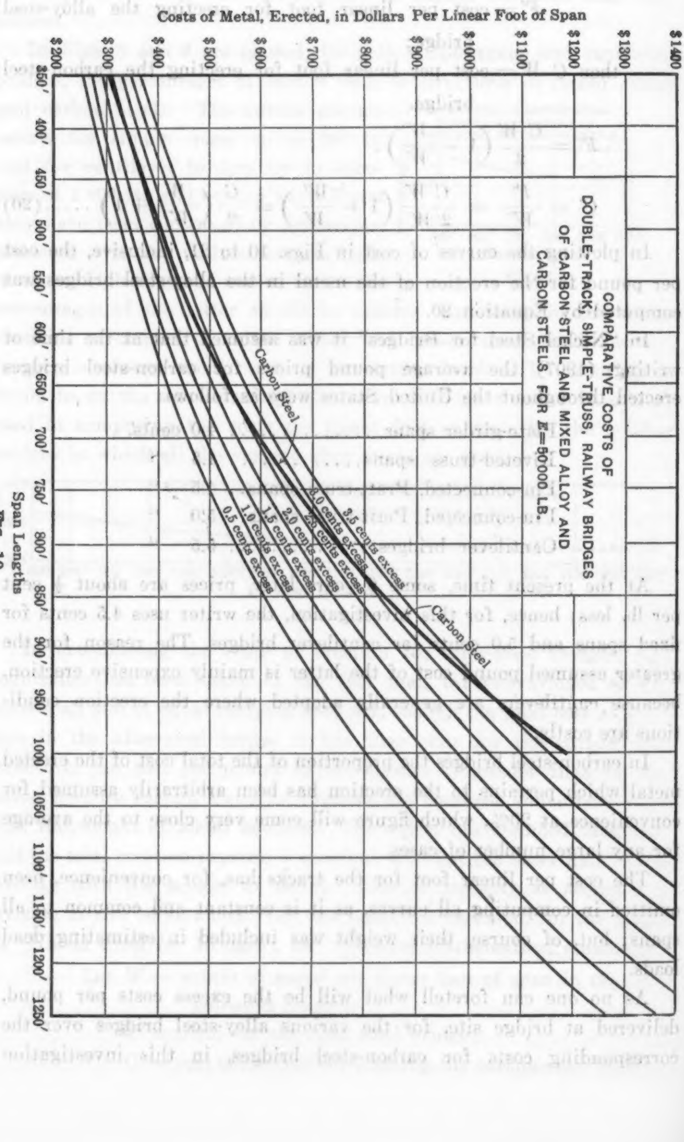

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=50000 LB.
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
Carbon Steel
350'
400'
450'
500'
550'
600'
650'
700'
750'
800'
850'
900'
950'
1000'
1050'
1100'
1150'
1200'
1250'
Span Lengths

Fig. 10.

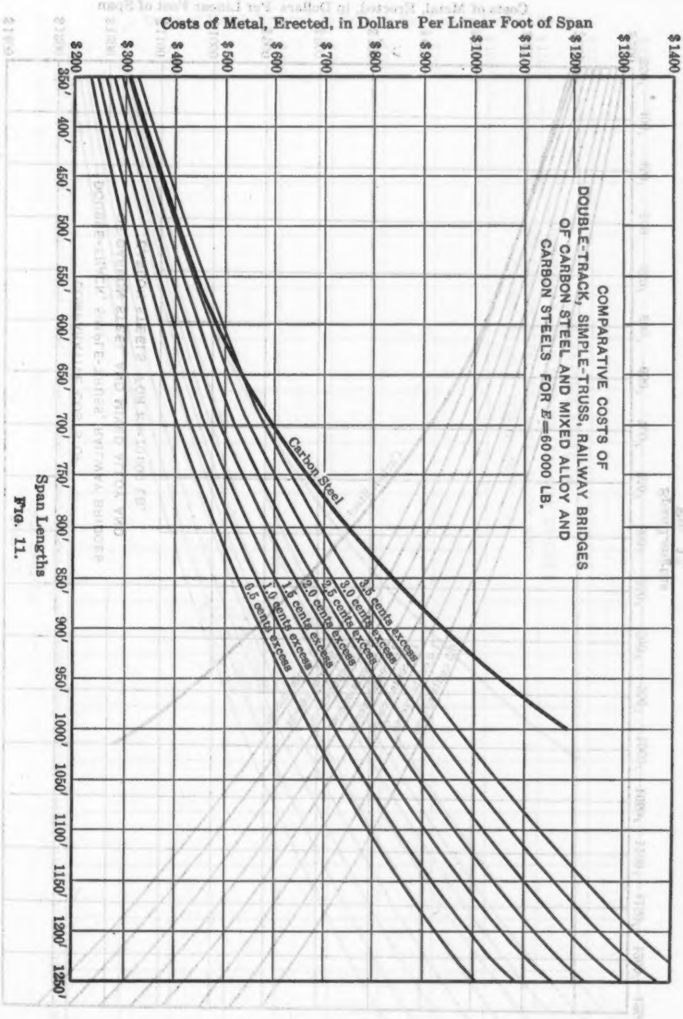

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=60000 LB.
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

Fig. 11.

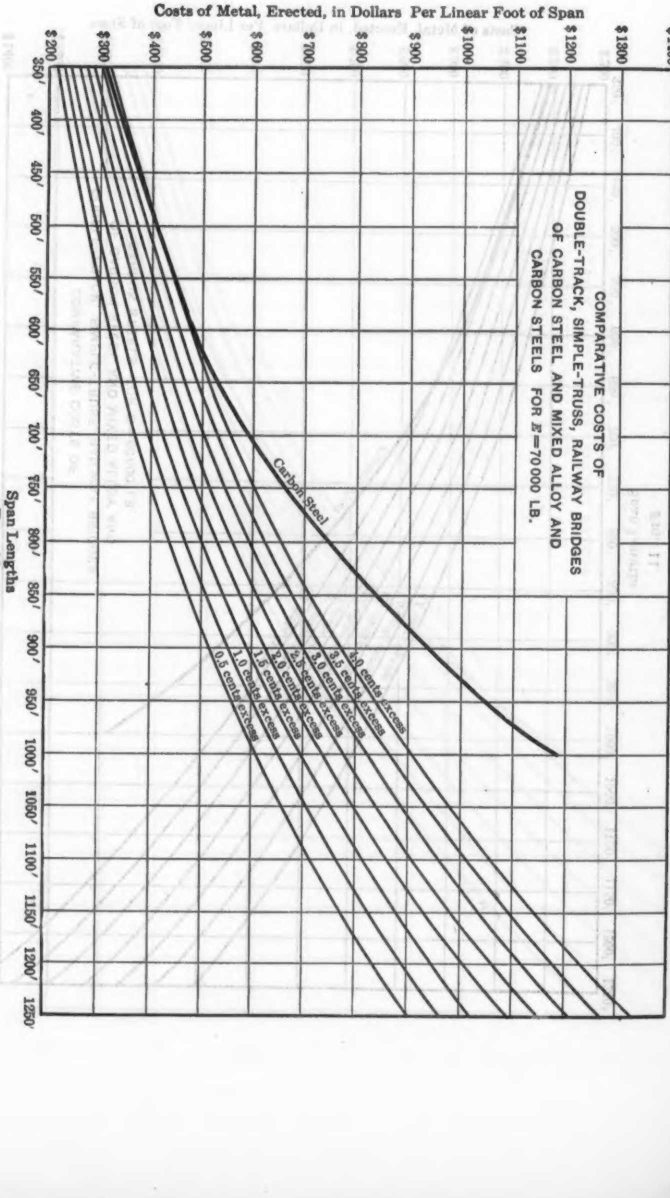

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=70000 LB.
Carbon Steel
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

Fig. 12.

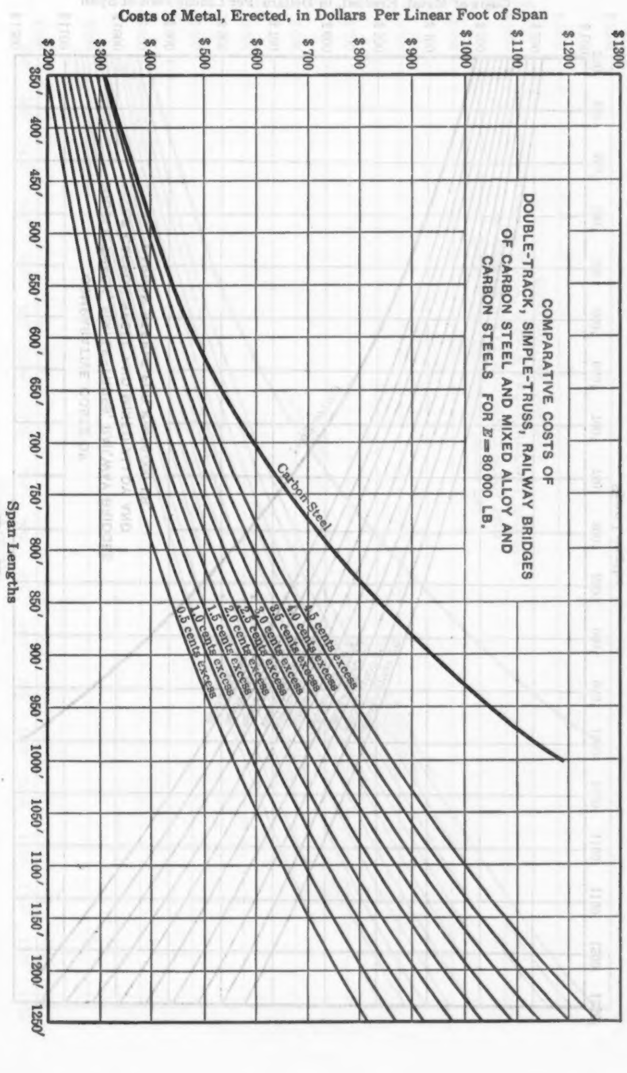
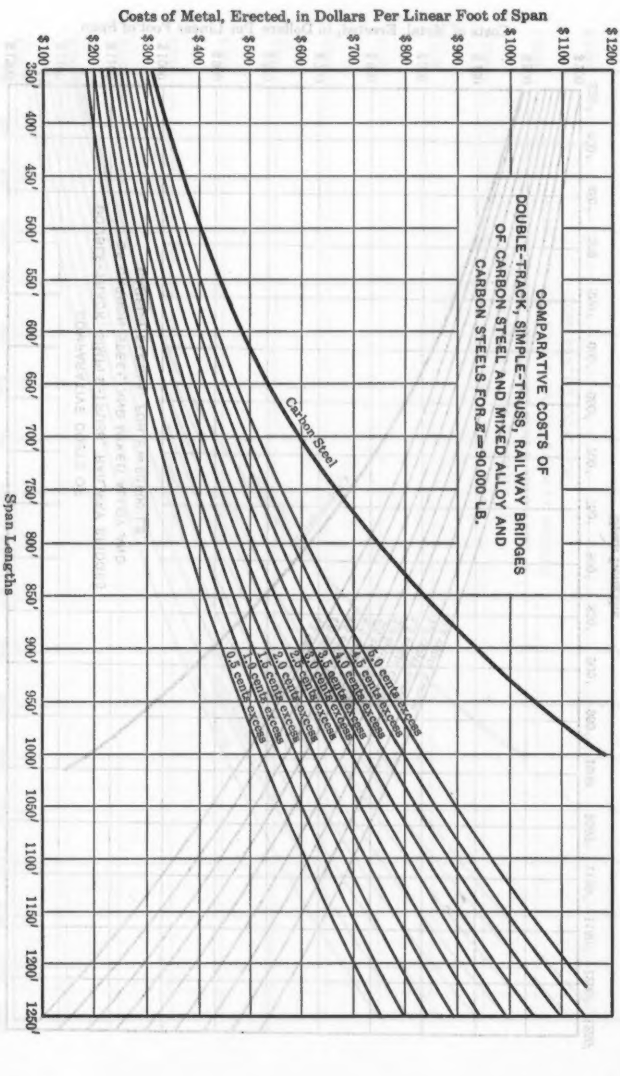

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$1300
$1200
$1100
$1000
$900
$800
$700
$600
$500
$400
$300
$200
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=80000 LB.
Carbon Steel
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350' 400' 450' 500' 550' 600' 650' 700' 750' 800' 850' 900' 950' 1000' 1050' 1100' 1150' 1200' 1250'
Span Lengths

FIG. 13.



Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=90 000 LB.
Carbon Steel
5.0 cents excess
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

FIG. 14.

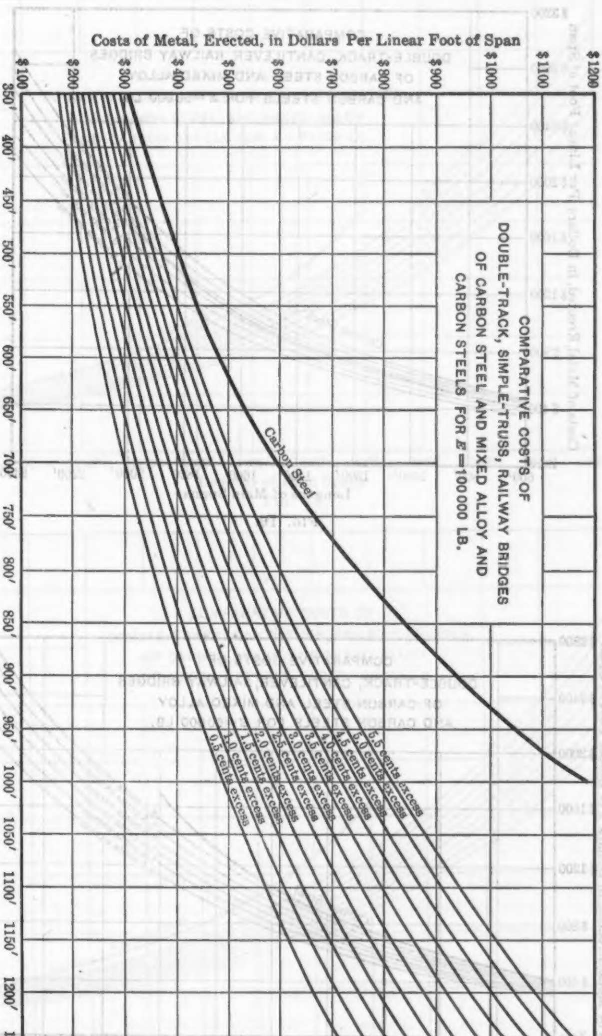

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E =100000 LB.
Carbon Steel
6.5 cents excess
6.0 cents excess
5.5 cents excess
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350'
400'
450'
500'
550'
600'
650'
700'
750'
800'
850'
900'
950'
1000'
1050'
1100'
1150'
1200'
1250'
Span Lengths

Fig. 15.

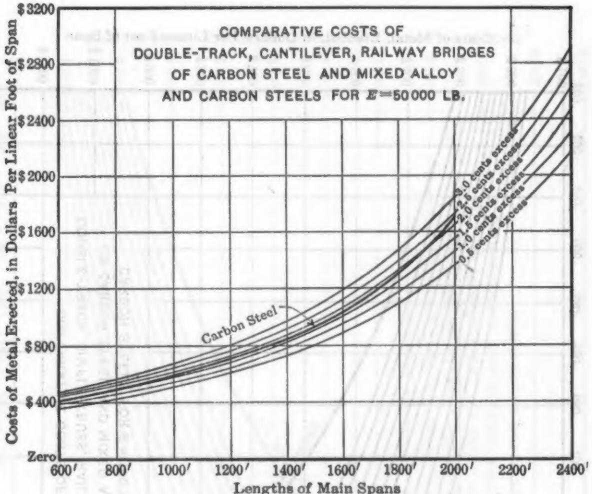

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=50000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′
800′
1000′
1200′
1400′
1600′
1800′
2000′
2200′
2400′
Lengths of Main Spans
Carbon Steel
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

FIG. 16.

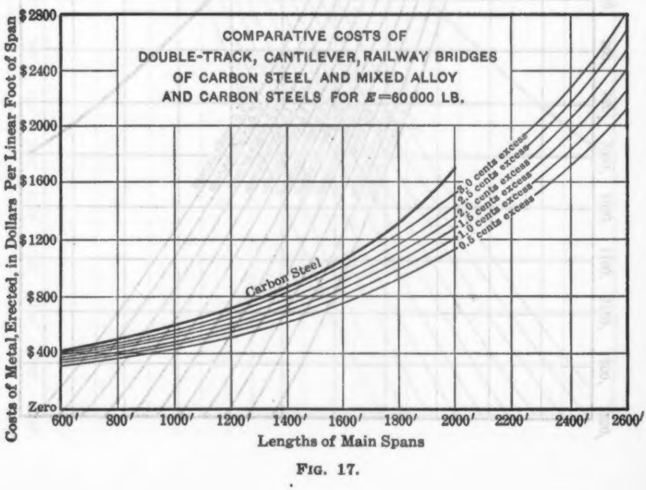

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=60000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′
800′
1000′
1200′
1400′
1600′
1800′
2000′
2200′
2400′
2600′
Lengths of Main Spans
Carbon Steel
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

FIG. 17.

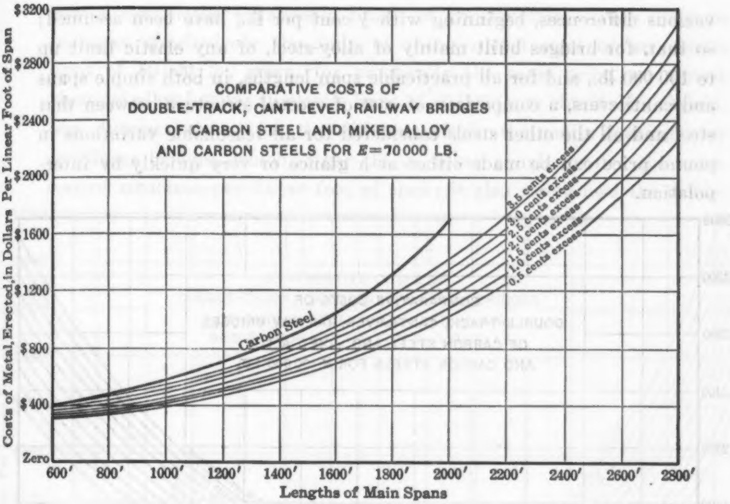

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=70000 LB.
Costs of Metal, Erected in Dollars Per Linear Foot of Span
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′ 2800′
Lengths of Main Spans
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

FIG. 18.

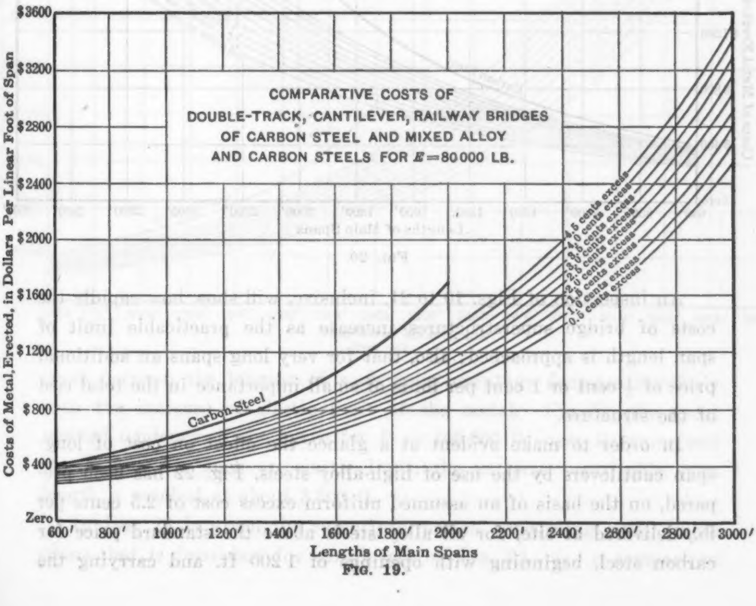

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=80000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3600
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′ 2800′ 3000′
Lengths of Main Spans
Carbon Steel
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

FIG. 19.

various differences, beginning with ½ cent per lb., have been assumed; so that, for bridges built mainly of alloy-steel, of any elastic limit up to 100 000 lb., and for all practicable span lengths, in both simple spans and cantilevers, a comparison of cost of erected structure between that steel and all the other steels considered for all reasonable variations in pound price can be made either at a glance or very quickly by interpolation.

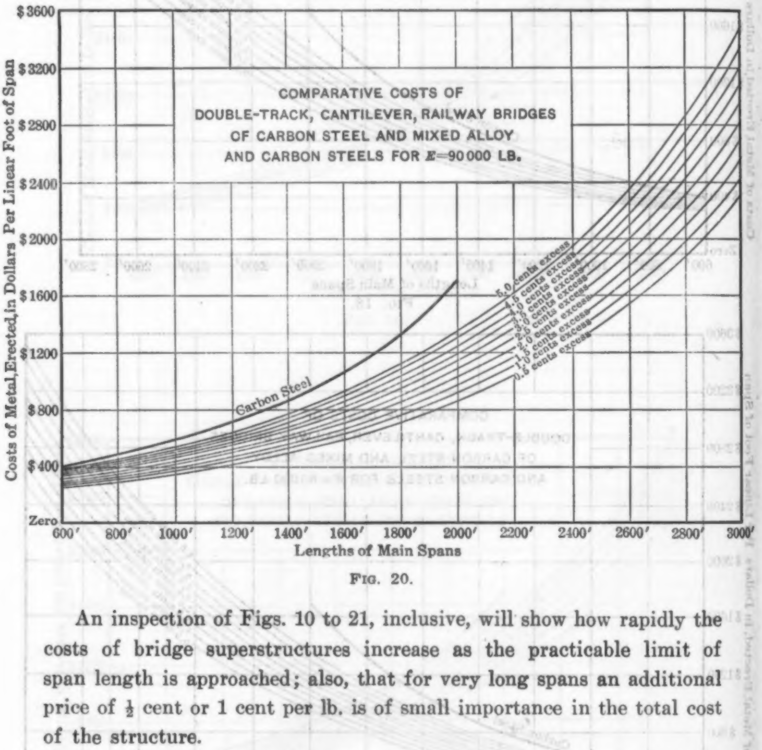


FIG. 20.

An inspection of Figs. 10 to 21, inclusive, will show how rapidly the costs of bridge superstructures increase as the practicable limit of span length is approached; also, that for very long spans an additional price of ½ cent or 1 cent per lb. is of small importance in the total cost of the structure.

In order to make evident at a glance the effect on cost of long-span cantilevers by the use of high-alloy steels, Fig. 22 has been prepared, on the basis of an assumed uniform excess cost of 2.5 cents per lb., delivered at site, for all alloy steels above the standard price for carbon steel, beginning with openings of 1 200 ft. and carrying the

curves out to the practicable limits of construction. This diagram indicates conclusively the folly of using nickel steel of an elastic limit of 50 000 lb., when, for practically the same price, an almost identical alloy having an elastic limit of 60 000 lb. is obtainable. Again, this diagram shows, for the different steels, the main openings for cantilever bridges, which, other things being equal, involve the same cost of structure per linear foot of span; it also shows how the differ-

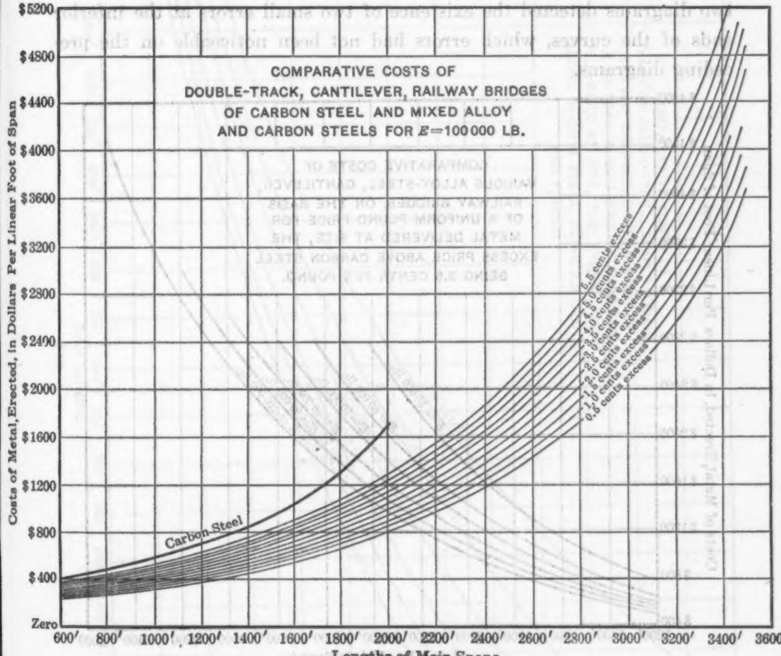


Fig. 21.

ences between the lengths of such main openings decrease regularly with the increase of elastic limit of the metal. Finally, the great upward tendency of the curve for $E = 100\,000$ lb. near its outer end shows that, for such an elastic limit, the extreme practicable span length is reached at about 3 500 ft.

In Fig. 23 is given the same information concerning simple-truss spans that is furnished for cantilevers by Fig. 22, *viz.*, a comparison

of costs of metal per linear foot of span for all the alloy steels, on the basis of adopting a uniform excess price over carbon steel of 2.5 cents per lb.

Incidentally, Figs. 22 and 23 afford an excellent check on the correctness of the writer's numerical computations; for, if there were any ordinary error made, it would be indicated at once by irregularity, either in the curves or their spacing; in fact, the plotting of these two diagrams detected the existence of two small errors at the inferior ends of the curves, which errors had not been noticeable on the preceding diagrams.

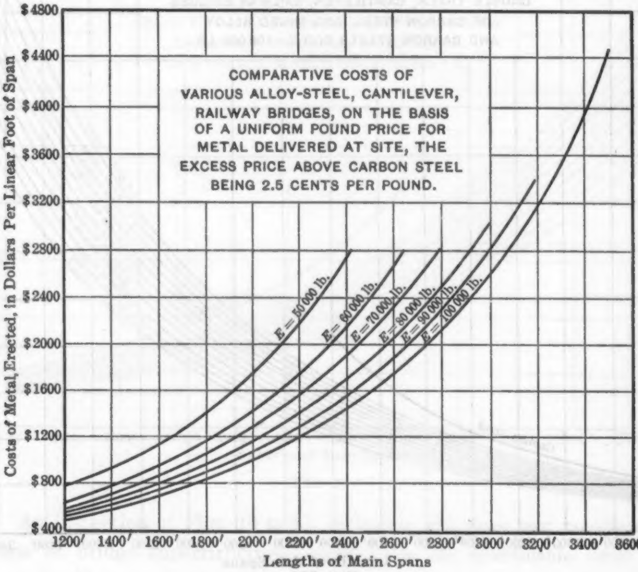


FIG. 22.

This brings up the question of the correctness of all the work in this investigation. There can exist nowhere any small errors of any importance, because they would have been detected at once by the lack of continuity in the diagrams. This feature, though, would not prevent the existence of fundamental errors based on incorrect assumptions or on wrong primary data. Such fundamental errors, however, are really impossible, because the weights of carbon-steel bridges dia-

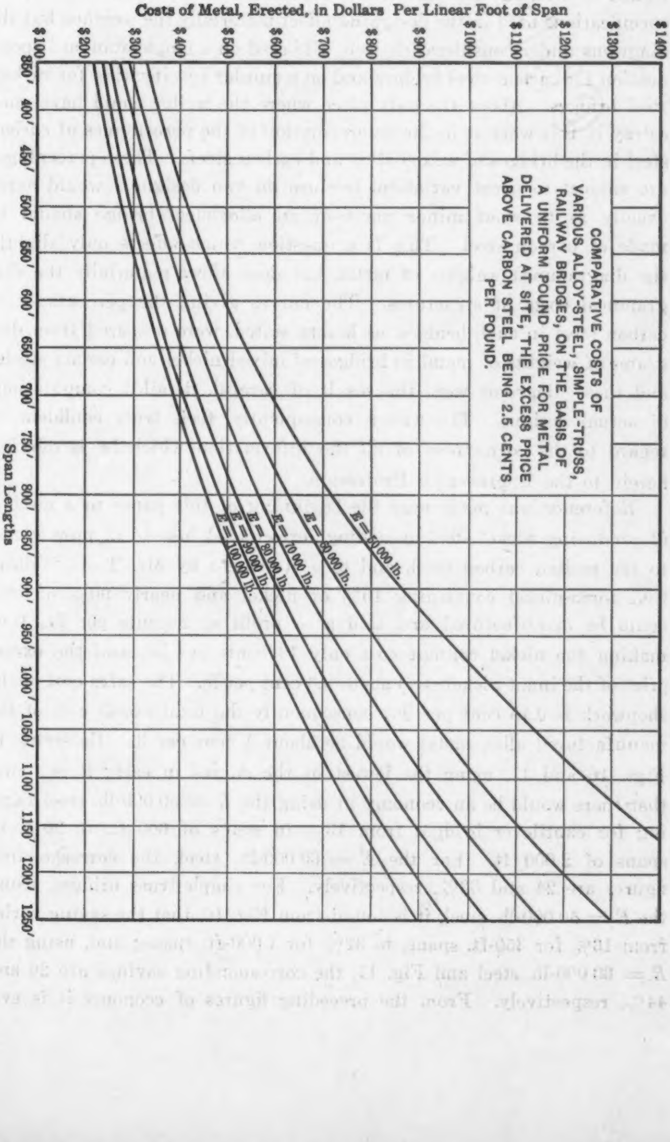

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF
VARIOUS ALLOY-STEEL, SIMPLE-TRUSS,
RAILWAY BRIDGES, ON THE BASIS OF
A UNIFORM POUND PRICE FOR METAL
DELIVERED AT SITE, THE EXCESS PRICE
ABOVE CARBON STEEL BEING 2.5 CENTS
PER POUND.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.
350'
400'
450'
500'
550'
600'
650'
700'
750'
800'
850'
900'
950'
1000'
1050'
1100'
1150'
1200'
1250'
Span Lengths

Fig. 23.

grammed in the writer's office are very exact. Of course, the personal equation of the designer affects the weight of metal in structures designed by him, but this variation is confined to small limits. Again, the specifications used in the designing affect materially the weights, but the diagrams under consideration were all based on a single standard specification for carbon-steel bridges and on a similar specification for nickel-steel bridges. About the only place where the writer could have gone astray in this work is in the determination of the percentages of carbon steel in the bridges of mixed alloy and carbon steels. These percentages are subject to great variation, because no two designers would agree exactly as to what minor parts of an alloy-steel bridge should be made of carbon steel. This is a question which affects only slightly the diagrammed weights of metal, but does affect materially the diagrammed costs of structures. The curves giving the percentages of carbon steel in such bridges, as before stated, were prepared from diagrams of weights of metal in bridges of mixed nickel and carbon steels; and these diagrams were the result of careful, detailed computations of actual designs. The writer, consequently, feels truly confident in regard to the correctness of all the information which he is offering herein to the Engineering Profession.

Reference was made near the beginning of this paper to a method of producing nickel steel by adding ferro-nickel instead of pure nickel to the molten carbon steel, and to a statement by Mr. T. L. Willson that ferro-nickel containing 10% of nickel and nearly 90% of iron could be manufactured and sold at a profit at 2 cents per lb., thus making the nickel content cost only 10 cents per lb., and the excess price of the ingot nickel steel about 0.3 cent per lb. The extra cost of the shopwork is 0.15 cent per lb.; consequently the total excess cost of the manufactured alloy metal would be about ½ cent per lb. Referring to Figs. 16 and 17, using the lowest of the curves in each, it is found that there would be an economy in using the $E = 50\,000$-lb. steel varying for cantilever bridges from 13% in spans of 600 ft. to 20% in spans of 2 000 ft. For the $E = 60\,000$-lb. steel, the corresponding figures are 24 and 33%, respectively. For simple-truss bridges, using the $E = 50\,000$-lb. steel, it is found from Fig. 10, that the saving varies from 16% for 350-ft. spans, to 32% for 1 000-ft. spans; and, using the $E = 60\,000$-lb. steel and Fig. 11, the corresponding savings are 26 and 44%, respectively. From the preceding figures of economy it is evi-

dent that it would pay the builders of large bridges to experiment with the use of ferro-nickel in the manufacture of nickel steel for bridge building, so as to determine whether the claims made by Mr. Willson in regard to its use are borne out by the facts.

The weights of metal for double-track railway bridges given in Figs. 5 and 6 can be utilized in estimating costs of any long-span railway bridges, because, if there are more than two tracks, the weights of metal per linear foot of span will, *ceteris paribus,* be directly proportional to the number of tracks. This is because, if only two trusses be used, the small saving in metal, in the trusses and the lateral system, will be offset by the extra weight of the floor-beams; and, if more than two trusses are adopted (as would generally be the case so as to avoid truss members of excessive cross-section), the economy of metal would be but slight.

Should a different live load per track from those herein used be desired, the weight curves on the diagrams can be modified accordingly by using the formulas in Equations 3, 4, 10, and 12; but, to do this correctly, one would need to know the division of total weights of metal per linear foot of span between the four components, "Floor System", "Lateral System", "Trusses", and "On Piers." Although it is not practicable, on account of space restriction, to give in this paper such a division with great accuracy, Tables 1 to 6, inclusive, will enable any one to calculate approximately, for any length of span and any kind of bridge herein included, the division of metal required.

However useful, though, may be the information given concerning weights of metal per linear foot of span for bridges in general, the principal object of this paper is to indicate the possibilities in bridge construction that may be attained by the use of high-alloy steels, and it is evident that the results of the writer's computations clearly prove that a systematic series of experiments made in search of a suitable and satisfactory alloy would be well worth while. Already it is practicable to obtain plate and shape nickel steel of 60 000 lb. elastic limit and eye-bar nickel steel of 65 000 lb. elastic limit; and, in the writer's opinion, it would not take much experimenting to raise each of these figures 10 000 lb.; but, to attain an elastic limit of 100 000 lb. or even 90 000 lb. for an alloy steel suitable for all shop manipulations is truly a great problem, and one worthy of much effort and a large expenditure of time and money.

TABLE 1.—Floor System for Simple Spans.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.		
	For 350-ft. span.	For 600-ft. span.	For 1 000-ft. span.
Carbon steel	1 400	1 550	2 000
E = 50 000 lb.	1 150	1 300	1 750
E = 60 000 "	1 000	1 150	1 600
E = 70 000 "	900	1 050	1 500
E = 80 000 "	850	1 000	1 400
E = 90 000 "	800	950	1 300
E = 100 000 "	750	900	1 200

TABLE 2.—Lateral System for Simple Spans.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.		
	For 350-ft. span.	For 600-ft. span.	For 1 000-ft. span.
Carbon steel	450	600	1 200
E = 50 000 lb.	450	600	1 150
E = 60 000 "	450	600	1 100
E = 70 000 "	450	600	1 050
E = 80 000 "	450	600	1 000
E = 90 000 "	450	600	950
E = 100 000 "	450	600	900

TABLE 3.—On Piers for Simple Spans.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.	
	For 350-ft. span	For 1 000-ft. span.
Carbon steel	250	400
E = 50 000 lb	200	300
E = 60 000 "	190	280
E = 70 000 "	180	260
E = 80 000 "	170	240
E = 90 000 "	160	220
E = 100 000 "	150	200

The writer is of the opinion that the first step to take is to experiment on "purified" steel, so as to bring it to its maximum of effectiveness, then to try adding nickel in various quantities, and afterward nickel and other, but cheaper, substances. Of course, augmenting the quantity of carbon in the purified steel, while increasing both its

TABLE 4.—FLOOR SYSTEM FOR CANTILEVER BRIDGES.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800-ft. span.	2 400-ft. span.	3 000-ft. span.	3 600-ft. span.
Carbon steel........	1 600	1 800	2 000	....	....	....
E = 50 000 lb........	1 200	1 450	1 650	2 400	....	....
E = 60 000 "	1 000	1 200	1 400	2 100	....	....
E = 70 000 "	950	1 150	1 300	1 850	....	....
E = 80 000 "	900	1 050	1 200	1 700	2 200	....
E = 90 000 "	850	950	1 100	1 600	2 000	....
E = 100 000 "	800	900	1 000	1 500	1 900	2 400

TABLE 5.—LATERAL SYSTEM FOR CANTILEVER BRIDGES.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800-ft. span.	2 400-ft. span.	3 000-ft. span.	3 600-ft. span.
Carbon steel.........	800	1 100	1 400	....	....	....
E = 50 000 lb........	800	1 050	1 300	1 800	....	....
E = 60 000 "	800	1 000	1 200	1 600	....	....
E = 70 000 "	800	1 000	1 200	1 600	2 000	....
E = 80 000 "	800	1 000	1 200	1 600	2 000	....
E = 90 000 "	800	1 000	1 200	1 600	2 000	2 400
E = 100 000 "	800	1 000	1 200	1 600	2 000	2 400

TABLE 6.—ON PIERS FOR CANTILEVER BRIDGES.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800 ft. span.	2 400-ft. span.	3 000-ft. span.	3 600-ft. span.
Carbon steel........	700	1 100	2 100	....	....	....
E = 50 000 lb	600	900	1 700	....	....	....
E = 60 000 "	580	860	1 600	2 200	....	....
E = 70 000 "	560	820	1 500	2 100	3 500	....
E = 80 000 "	540	780	1 400	2 000	3 200	....
E = 90 000 "	520	740	1 300	1 900	2 900	3 900
E = 100 000 "	500	700	1 200	1 800	2 600	3 600

ultimate strength and its elastic limit, would tend to harden the metal; but the addition of nickel (and possibly other elements) would tend to reduce the brittleness and render it workable.

The problem of finding a high, cheap alloy of steel, suitable in every particular for bridges, is now before the metallurgists and the

builders of large metallic structures; and the values of all the results probably attainable are clearly indicated in this paper; hence the onus is on the Engineering Profession to see that the necessary experiments are arranged for and thoroughly carried out, in order that the world may have at its command a new metal that will permit of the spanning of waterways which are so wide and so deep, or are so restricted by navigation requirements, as at present to defy the art of the bridge engineer.

DISCUSSION

M. J. BUTLER,* M. AM. SOC. C. E. (by letter).—The author has once more placed the members of the Society under obligations. He has dealt most ably with a matter of great interest and importance. Mr. Butler.

It is an error to assume that the limit has been reached with ordinary carbon steel—undoubtedly, if the customer is willing to pay the price, the steel maker will supply a much better quality of steel:

(*a*) By fluid compression of the ingots;

(*b*) By careful heat treatment and lighter and slower blooming, putting more work on the bloom;

(*c*) Cropping away all segregated, piped, and inferior metal;

(*d*) Careful re-heating of billets, slower rolling, and better work generally on the rolls and sections;

(*e*) Such a quality of acid steels—possibly with a cheaper alloy than nickel—would allow of higher unit stress in large members, where shock and impact would be amply cared for.

Alloy steel of nickel and other materials may be secured to meet any reasonable requirements. Steel makers will rise to the occasion, when the demand justifies the outlay in getting ready for steel of such a quality.

In very great bridges, such as are under consideration in this paper, the increased price of the metal would not be so serious; there are too many other factors that go to make the cost, such as the substructure, the erection of great cantilevers, and the special tools, plant, and shop, to fabricate the member. The engineering and the financial costs are to a considerable extent independent.

ALBERT LUCIUS,† M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest. Its objects are stated to be: First, to give the weights of different span lengths for simple spans and cantilever bridges, using metals of various elastic limits; secondly, to indicate the extreme practical limits of span lengths of cantilever bridges constructed of such metals; and, thirdly, to demonstrate the comparative economics of finished bridges involved in using metals of the various assumed elastic limits. Mr. Lucius.

Concerning the weights of long-span bridges built of assumed metals of higher elastic limits, whatever their actual amount may be, there is little doubt that weights would reduce about as formulated in the paper if the unit stresses follow the elastic limits, as they should, provided the metals have the same superior properties as to ductility which they have as to strength, for then only would confidence in the material be justified to the extent of developing all sections and all details carrying calculated stress by the proportional unit

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† New York City.

Mr. Lucius. strains. Members and their connections and all details, the function of which is to distribute local strains over sectional areas, could then be made of the same material working under the same stress and elastic condition, but, if the high alloys are of inferior ductility and connections and distributing details must be made of softer and weaker metals, and the unit amounts of extension and compression vary in the different parts forming the ends of a member, the difficulties of making proper connections would be increased and the advantages of reduced main sections would be partly neutralized.

Concerning the extreme length of cantilever spans, it is the writer's opinion that it has been reached, as far as practicable; that it will not be carried very much further, even if higher grade material is available; and that every consideration of reliability to carry load, simplicity of the carrying member, availability of the highest grade of material, and ease of construction and erection, points to the suspension bridge as the type to take care of extreme span lengths where bridges are preferred to subway constructions.

Concerning the economics involved in the use of high elastic limit materials, these are expressed most readily in a sufficiently satisfactory way by the ratio of elastic limit stress to price per pound of the metal, and whichever metal sells at the least price for the highest elastic limit, all other properties being in proper relation to it, is the cheaper material, and this would apply to the various types of bridges alike as far as their fabricated cost is concerned. Generally, the idea of using simple spans up to their extreme limit of possible application cannot be attractive to engineers, because there is hardly any other reason for long spans than the requirement to maintain an unobstructed right of way under these proposed spans. The feasibility and cost of erection will determine the type, and a cantilever or continuous span over several openings, or an arch bridge, all of which can be erected cantilever fashion, would probably be selected before the economic limit of the fabricated simple-span bridge is reached. Besides, in long-span bridges, especially where prominently placed, esthetic considerations do and should enter more emphatically and obscure the importance of relative economics still further.

Concerning the desirability of improving on the present available bridge material, both as to uniformity of present properties and in regard to increase of all its elastic properties and its ductility, there is probably no difference of opinion among engineers, and each, no doubt, does all he can to encourage and help push manufacturers to favorable solutions; but this is another subject, and in the care of the metallurgist. It is hardly likely that it is inertia alone which makes progress slow and apparently does no more than maintain a stationary level. It is more likely that there is a greater possibility of making economic improvements on the present grade of steel by

cheapening it and making it more uniform in quality, than to make a higher grade of steel at a price which will give it an advantage commercially, except for special purposes, in which case the price is not so important relatively to the whole special purpose to be accomplished. The vast majority of railroad bridges can be well taken care of with the present grade of steel, and whether lighter sections of higher grade steel worked to higher strains would be an all-round improvement for the bulk of bridgework, might be open to debate. It certainly would not be the case, in the writer's judgment, if higher strength did not also carry with it higher ductility. Mr. Lucius.

The engineer must get next to the metallurgist. It is highly probable that any metallurgist who knows how to produce a better steel alloy at commercial value will succeed very promptly in getting it on the market, and engineers would accept it as promptly and adjust their constructions to it.

Henry W. Hodge,* M. Am. Soc. C. E.—The length of bridge spans in general use has been increasing steadily, and we have reached limits where the dead weight of the structure has become the largest portion of its carrying capacity, so that some method of keeping the weight down is a necessity for the construction of the great spans now contemplated. The only way to reduce the dead load materially is by the use of metal of higher carrying capacity than our present materials, and a long step in this direction has been made by the use of nickel steel, which has 50% greater carrying capacity than the carbon steel in general use. Mr. Hodge.

The trusses of the three 668-ft. spans of the St. Louis Bridge were designed for nickel steel throughout, except certain minor sub-members. Nickel-steel eye-bars and carbon-steel compression members were also used, the floor system and bracing being of carbon steel in each case.

The weights of each span were:

With complete nickel-steel trusses........	9 200 000 lb.
With nickel-steel bars, and the rest of carbon steel	10 900 000 lb.

The dead load of railways, tracks, etc., was 5 500 lb. per lin. ft., so that the total average dead load was:

With nickel-steel trusses...........	19 300 lb. per lin. ft.
With nickel-steel bars, and the rest of carbon steel	21 800 lb. per lin. ft.

Thus, the nickel-steel compression members in the trusses made a difference in weight of 2 500 lb. per lin. ft., or 13 per cent.

The average live load on the two decks was equal to 16 600 lb. per lin. ft., thus the use of nickel-steel compression members made a saving of 7% in the total load on the structure.

* New York City.

Mr. Hodge. The average unit prices for the two classes of material in this structure, erected in place, were:

Nickel steel, 5.6 cents per lb.
Carbon steel, 3.95 " " "

making a difference of 1.65 cents, or 42% of the price of the carbon steel; but the elastic limit required for the nickel steel was 50% higher than for the carbon steel, so that the nickel steel was the cheaper, strength for strength.

This steel had $3\frac{1}{2}$% of nickel, but manufacturers are now commercially producing an alloy steel, with not more than $1\frac{1}{2}$% of nickel, together with small percentages of chromium and vanadium, which has all the properties of this steel, at a very much reduced price, so that there is at present a readily obtainable material, which is 50% stronger than the carbon steel in general use, at a comparatively small increase in cost.

This increase of elastic limit to 50 000 or 60 000 lb. per sq. in. will help greatly in the construction of spans of considerable length; but, for the very long spans now being planned, a still stronger material is needed, and can economically be used at a very considerable increase in price. In the design for the 2 880-ft. suspension span for the North River Bridge at New York, there is great economy in placing the stiffening trusses along the cables so that articulated joints are required, thus necessitating eye-bars in place of wire, which has heretofore been used in long suspension bridges.

The total live load on the eight tracks, two roadways, and two sidewalks, is 20 000 lb. per. lin. ft., when all are completely loaded; and, if the structure were made of carbon steel, the dead load would be about 110 000 lb. per lin. ft., which would make the sections almost prohibitive. The designs, therefore, have been based on the use of eye-bar cables of alloy, heat-treated steel, having an elastic limit of not less than 120 000 lb. Such steel has already been manufactured for limited sizes, and as it is only here required for eye-bars, from which any number of full-sized specimens can be tested to destruction, there will be no doubt as to whether the strength and other qualifications are fulfilled. With such material, the total average dead load of the structure is 64 000 lb. per. lin. ft., so that this material makes a saving of 46 000 lb. per lin. ft., or practically 50% of the total load on the structure.

Such a material will naturally cost a considerable price, but it will be about four times as strong as the usual carbon steel; and even at much more than four times the cost, it would make a saving, on account of the large decrease in dead load.

About 40 000 tons of such bars will be required, practically all duplicates, so that there is little doubt that they will be furnished at a price which will make a great economy in the structure. Mr. Hodge.

The speaker, therefore, fully agrees with Mr. Waddell that such high-value alloy steels are a necessity for coming bridge structures, and has the fullest confidence that our metallurgists will meet the demands as they arise.

CHARLES EVAN FOWLER,* M. AM. SOC. C. E. (by letter).—For many years the writer has known of the higher grades of steel produced by European manufacturers, but, owing to the great demand on the mills of the United States, it has been a case of taking what is offered by the manufacturers or paying the extras asked for nickel steel or anything except the ordinary grades. The tariff conditions will now allow the importation of European high-grade steels for Pacific Coast fabrication, but, until shops of sufficient size are established there, little advantage can be taken of such foreign products. This possible source of supply may eventually result in American mills being willing to meet the demand for nickel or other high steel alloys at a reasonable price. Mr. Fowler.

In the writer's discussion† of Mr. Waddell's former paper, "Nickel Steel for Bridges", he called attention to the possible use of ferruginous nickel as a cheap means of obtaining the nickel ingredient for nickel steel, and it is gratifying to know that this has been declared possible by competent metallurgists and at a reasonable cost; so that now it seems to be only necessary to find philanthropic steel manufacturers who will make only a reasonable charge for manufacturing plates and shapes of the desired composition.

To conduct the extensive experiments which are desirable on nickel steel and other high steel alloys, it will certainly be necessary to enlist a really enthusiastic support from the mills before anything satisfactory can be accomplished. When it is possible to do this, and to raise the necessary funds, the work should be carried out under the direction of this Society, acting in conjunction with the American Society for Testing Materials.

The great amount of work done by Mr. Waddell, as shown by this and his former paper, will entitle him to a large share of the credit for the great spans which may be built in the future if, as a consequence, higher steels are made possible.

The diagrams of weights of metal in bridges, for both the usual and the higher grades of metal, show conclusively what one would naturally infer as to the great saving that may be made by the use of a steel of high elastic limit which can be obtained at a reasonable cost. It is to be regretted that all such investigations are not made

* Seattle, Wash.

† *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 300.

Mr. Fowler. on a common basis, both as to loadings and specifications, so that the work of various engineers along similar lines may be more readily compared, and the desired end be more quickly reached. To this end, the proper committee of the Society should formulate rules for all such investigations and calculations.

The engineer who has been in direct touch, not only with structures manufactured from his own designs, but also with those of hundreds of other engineers, as has been true in the writer's case at the shops of the Youngstown Bridge Company and at other large shops, and throughout a wide experience of more than a quarter of a century, will realize that the personal equation of the designer does result in such a wide variation in weights that, unless each one is tied down to the hard and fast rules of the same specifications, the resulting data cannot readily be compared, especially for long spans.

The real comparison can only come from the various designs that would naturally be made for any specific location and the investigation of the composition of the necessary high-grade metal required for that particular structure, as was the case for the St. Louis Eads Bridge. A structure costing from $30 000 000 to $60 000 000 would easily stand the charge of $100 000 or more for such experiments.

Long spans are seldom contemplated or built from motives of economy, but are the result of the necessities of commerce, of finding good foundations, or as necessary connecting links in lines of traffic and communication, regardless of whether the structure in itself will be a paying investment. When, from some cause or other, long spans are found to be necessary, the engineer must determine the class of structure that will be possible:

First.—To carry the class and quantity of traffic to be imposed or cared for;

Second.—That will be possible, due to the foundations that can be obtained;

Third.—That will be possible, due to the material that can be obtained from which to fabricate it;

Fourth.—That will be possible, owing to the limitations that may be imposed by the methods and means of construction;

Fifth.—That will be a paying investment from a dividend-paying standpoint, if that be necessary;

Sixth.—That will at least be possible from related financial conditions.

These factors are correlated to such an extent that a very wide investigation would be necessary, in order to set the limits of span and expenditure.

Mr. Fowler.

Recent researches seem to indicate:

That simple spans can be constructed by using nickel steel to a greater or less degree, up to lengths of from two to three times those which have been built;

That suspension spans and cantilevers will reach about the same cost at some span length between 1 600 and 1 800 ft.;

That, by the use of nickel steel to a large extent, suspension spans can be built economically, where the traffic is sufficient, up to about 3 000 ft. span;

That, by the use of nickel steel to a large extent, cantilevers can be built economically, where the traffic is sufficient, up to about 2 500 ft. span.

However, inasmuch as the dead weight of such structures is very great in proportion to the live load, we can largely disregard impact, and use correspondingly high unit stresses, thus allowing at present cost the use of nickel steel for all the suspended structural parts of a suspension span, thereby making such a structure, with its high steel cables and comparatively low erection cost, the best to adopt for long spans, from every point of view, beyond a span length of about 1 700 ft. Such a structure will carry satisfactorily all classes of traffic, and satisfy most fully all the six requirements previously given.

The question of right of way and terminals may add so greatly to the cost of the project as to make its realization impossible.

In his discussion of Mr. Waddell's former paper, the writer called attention to the various cost factors that would come in to determine the possible span length of any type, or indeed the type to be used in a particular location. The sub-factors of erection cost will also serve to decide these things. The empirical erection cost formula, devised by the writer some years ago, will serve to illustrate this point, there being six different factors which enter into the cost of only this one portion of the amount necessary to expend for the construction of a bridge superstructure:

$$C = a + \sqrt{l} + \frac{3}{4}h + \frac{200}{d} + .5\,p - \frac{1}{5}\sqrt{w - 500}$$

C = cost of erection, in cents per 100 lb.;

a = a constant for each type of structure;
= 15 cents for railway pin trusses;
= 25 cents for railway riveted trusses;
= 20 cents for railway girders;
= 20 cents for highway pin trusses;
= 30 cents for highway riveted trusses;
= 15 cents for highway girders;

l = span length, in feet;

h = height of falsework, in feet;

d = daytime temperature, average, in degrees Fahrenheit.

Mr. Fowler.

Taking the case of a riveted railway span 225 ft. long, 48 ft. height of falsework, average temperature, 40°, 2 coats of paint, and weighing 2 100 lb. per lin. ft., we find the probable erection cost to be 83 cents per 100 lb., or $16.60 per ton.

A railway pin span of 144 ft., 36 ft. height of falsework, average temperature, 50°, 2 coats of paint, and weighing 1 400 lb. per lin. ft., erection cost equals 62 cents per 100 lb., or $12.40 per ton.

These two examples show what an influence a change in any one of the factors will have on the unit erection cost. For cantilevers of the type of the Poughkeepsie Bridge, or the Knoxville Bridge, designed and built by the writer (Fig. 24), every other span being a fixed or anchor span, the bridge must have falsework, so that, for a bridge of this type one can take the coefficient of h at only $\frac{3}{8}$ instead of $\frac{3}{4}$. The formula was deduced to fit certain conditions, which to a large extent were due to the personal equations of the designer and the erector; and, with plans prepared by some designers, the cost of erection would exceed very greatly the values found from the formula, it being only too common on the part of many designers to forget that structures must be erected at a reasonable cost, and still others seem to forget the process of erection entirely.

Many erection costs, of course, will exceed greatly what they should, due to unforeseen causes. The White Pass, Alaska, Arch constructed by the writer, was to have been completed before the winter season, but, owing to a strike of the workmen in September, and no telegraph line available to obtain a new crew from Seattle, 1 000 miles distant, some 6 weeks were lost, and the structure was completed by the men working in two shifts of 2 hours each, one crew warming up in camp while the other worked. The arch was completed in December, and is shown in Fig. 25. Had it not been for the delay and consequent extra cost, the erection could have been carried out on a very economical basis, as the falsework necessary for the anchor arms was very slight, and one top traveler carried out the entire work of erection.

The extra cost due to delay in this or other cases, cannot, of course, be considered in making a design, but there are many features of erection methods which should be studied out during the original investigations. So far as the writer is aware, no scheme of erection was ever formulated for the construction of the Williamsburgh Bridge over the East River, on which he was Consulting Engineer for the erection of Manhattan tower and approaches. The Brooklyn tower erection had been planned and partly carried out by constructing trusses across from masonry pier to masonry pier, and these supported a temporary combination tower about 300 ft. high from which to erect the steel tower.

FIG. 24.—KNOXVILLE STEEL ARCHED CANTILEVER BRIDGE.

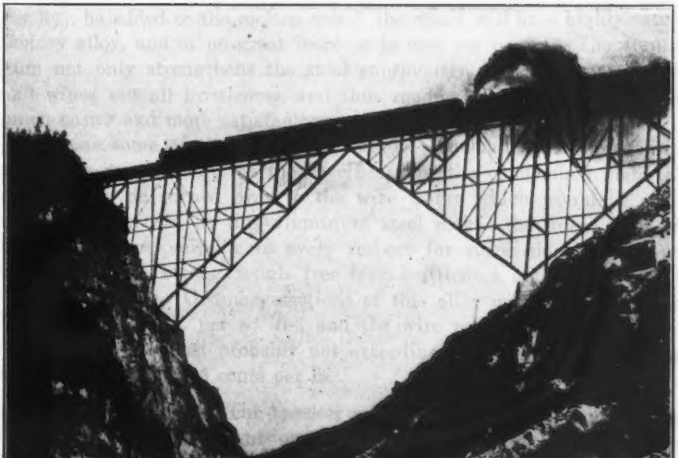

FIG. 25.—SWITCHBACK ARCH. WHITE PASS AND YUKON RAILWAY, ALASKA.

Mr. Fowler.

For the Manhattan end the tower was erected up to the floor level by stiff-leg derricks on a heavy bent at the end of the approach-span falsework. The latticed strut and floor-beam girder in the tower was also placed, and on this a timber tower about 150 ft. high was constructed to use in erecting the remainder of the tower steel, thus saving some thousands of dollars in erection cost.

The foregoing will serve to make perfectly plain the writer's reasons, in some few respects, for regarding diagrammed weights and costs as only very general proof of the necessity, in any particular case, for the use of the highest obtainable grade of steel. There are items in every part of the cost, of both substructure and superstructure of any great bridge, which will vary widely from similar items of cost in another large structure, and it is only by extensive investigations at each particular location that we can arrive at a close semblance of the truth and avoid the errors which often wreck meritorious projects.

Mr. Le Conte.

L. J. Le Conte,* M. Am. Soc. C. E. (by letter).—The writer is highly pleased with this valuable paper. The practical results given are exceedingly encouraging for future developments. The writer would respectfully suggest to the experimenters that they also try a small batch of aluminum steel. The main objection to its use, heretofore, has been the high market price. Lately, however, the price has come down to 20 cents per lb. in large lots. It is his firm belief that if the ordinary steel is first "thoroughly purified", and its elastic limit thereby raised to 60 000 lb. per sq. in., and then a cake of aluminum, say 3%, be added to the molten metal, the result will be a highly satisfactory alloy, and at no great increase in cost per pound. The aluminum not only strengthens the steel enormously, but also toughens it and wipes out all brittleness, and thus renders all general shop work much easier and more satisfactory and reliable in every way.

In case some of the material should be condemned as not coming up to the requirements of the specifications, the condemned material could easily be turned over to the wire works which would be only too glad to get it, as this aluminum steel makes the finest kind of high-grade wire, suitable in every respect for suspension bridges, as it is exceedingly strong, tough, free from brittleness, and non-corrosive to a high degree. Ordinary sections of this alloy will have an elastic limit of 100 000 lb. per sq. in., and the wire possibly 200 000 lb. per sq. in.; and at a cost probably not exceeding that of ordinary carbon steel by more than 2.5 cents per lb.

Mr. Petinot.

N. Petinot,† Esq.—The speaker agrees with Mr. Waddell that the first step is to experiment on the purification of steel so that the metal may be brought to its maximum efficiency. He cannot see

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† Metallurgist, The Titanium Alloy Manufacturing Company, Niagara Falls, N. Y.

Mr. Petinot. why it should not be possible to secure regularly physical properties within a limit of 5% in a steel carrying predetermined quantities of carbon and nickel, for instance.

The speaker has been experimenting extensively, both in France and in the United States, with the manufacture of alloy steels, and particularly with nickel steels of the same supposed analysis, and has found that very often two melts of open-hearth nickel steel of the same analysis, when physically tested, have shown a variation of 15% or more. Carrying the investigation further he found, by microscopic examination, a large difference between the two steels. The poorer very often showed very large colonies of manganese sulphides, iron silicates, and slag. These are weakening elements in steel of any grade, even if it is an alloy of plain carbon metal.

When the manufacturers of steel find the way to remove such elements, it will be possible to secure the maximum physical qualities from each component of every grade of steel, and there should be no great difference between any two steels of the same chemical composition.

Increasing the carbon content in steel of any grade is a cheap way to increase the alternate strength, and, according to the speaker's experience, a satisfactory way, provided the steel has been thoroughly cleaned and the segregation reduced to the minimum.

In reference to the possibility of producing a nickel steel by the use of a ferro-nickel containing 10% of nickel, this problem presents no difficulties at all, as such steels have already been produced in 5-ton melts. A few years ago a pig iron was produced by smelting pyrrhotite—containing nickel—in an electric furnace. This pig iron contained from 4 to 5% of nickel and was low in sulphur and phosphorus. By varying the proportions of such pig iron in the mixture used in making nickel steel by the open-hearth process, it will be possible to obtain any nickel content actually desired.

It will be noticed that, in the making of nickel steel, the nickel is usually added with the cold charge, and does not, therefore, oxidize during the periods of melting and finishing the steel, and its use as a component of pig iron would be the same under similar conditions.

The speaker believes that, by experimenting thoroughly with ferro-nickel in the manufacture of nickel steel, it will be possible to get a product which will be identical with that obtained by the use of metallic nickel, and at a much lower cost.

Mr. Waddell has asked what effect the use of titanium might be expected to have on the steel. Titanium has a great affinity for nitrogen and oxygen, so much so that, prior to this time, it has been very difficult to obtain a steel with a content of more than 0.25 of that metal, and such steel showed no particular advantages over one

which had been treated with 0.10 of titanium, and after such treatment carried only 0.02 to 0.03 of the latter. Mr. Petinot.

Titanium is on the market commercially in the form of ferro-carbon-titanium, containing from 15 to 20% of titanium and from 6 to 8% of carbon. It has been found that a ferro-titanium with a higher titanium content has too high a melting point for general use.

Titanium, because of its affinity for nitrogen and oxygen, the latter either in the form of a gas or an oxide, is to be considered as a powerful deoxidizer and scavenger. It is the only deoxidizer known at present which can be used without danger of leaving any of the products of its oxidation in the bath of steel.

In the treatment of steel with titanium, the ferro-carbon-titanium is always used as the last addition to the steel after it has been tapped into the ladle, that is, after manganese, silicon, or other alloys, if any, have been added. Titanium will react first on nitrogen (always present in a greater or less quantity), forming titanium nitrides (Ti_2N_2), which rise to the top of the ladle. It will then act on the oxygen in solution in the steel, and on oxides of iron, manganese, etc.

Particular attention is called to the presence of manganese oxide in steel. Ferro-manganese is added to the steel, not only as a deoxidizer, but also to furnish sufficient manganese to combine with the sulphur, giving manganese sulphides, which are less brittle than iron sulphides. It has always been found that the rolling properties of steel are enhanced by a small manganese content, and it is a well-known fact that it is impossible to roll a steel ingot which does not contain manganese.

Now consider, for instance, what happens when manganese is added to molten steel. The iron oxides, for instance, are robbed of their oxygen, and manganese oxides are formed, as shown by the following chemical equation:

$$FeO + Mn = Fe + MnO,$$

but, as manganese oxide will partly remain in the steel, if a piece of this steel is analyzed, the chemical laboratory will report so much manganese, but there will be nothing to show in what form this manganese occurs, whether as an oxide, sulphide, or some other compound.

It is difficult to determine whether a remainder of manganese oxide in the steel is more or less harmful than a content of iron oxide, which it may have replaced. In the manufacture of soft steel, with a carbon content of from 0.08 to 0.10 and manganese content of 0.40, it has been proved that, by eliminating the manganese oxide completely, no blisters will be found when such steel is rolled into sheets and galvanized.

When titanium is added to steel it will react as shown by the following equations:

Mr. Petinot.

On nitrogen:

$$Ti_2 + N_2 = Ti_2N_2 \ldots\ldots\ldots\ldots (1)$$

On oxygen:

$$Ti + O_2 = TiO_2 \ldots\ldots\ldots\ldots (2)$$

On iron oxide:

$$2FeO + Ti = 2Fe + TiO_2 \ldots\ldots\ldots\ldots (3)$$

On manganese oxide:

$$2MnO + Ti = 2Mn + TiO_2 \ldots\ldots\ldots\ldots (4)$$

The titanic oxide (TiO_2) formed by any of the last three reactions will flux all particles of slag invariably suspended in the steel and carry such slag to the top of the ladle, which will enable steelmakers to teem into the ingots a product practically free from nitrogen oxides and slag, which are the weakening elements in steel of every grade.

Another function of titanium is to reduce segregation to its minimum. The following explains briefly how segregation is produced: Steel must be considered as a mixture of pure iron, iron carbide, manganese sulphides, iron silicides, iron phosphides, etc., each of the components having different specific gravities, and varying melting points. Now, consider what will happen when an ingot mould is filled with molten steel: The wall of the mould will act as a chill, and the component having the highest melting point will be the first to solidify. If drillings are taken, starting from the outside of the ingot toward its center, it will be found that the carbon and other elements of lower specific gravity and higher melting points will be higher inside the part cross-hatched in Fig. 26 and also at the top of the ingot.

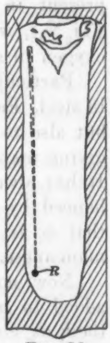

FIG. 26.

Now, consider why the various elements composing the steel will segregate, and why the maximum segregation will be found at the top of the ingot. This segregation is produced by the presence of oxides in the steel, no matter whether they are of manganese, of iron, or other elements. Take, for instance, a molecule of oxide, *R* (Fig. 26). The carbon of the steel will react on this molecule, giving carbon monoxide, according to the following equations:

(In the case of iron)

$$FeO + C = Fe + CO \ldots\ldots\ldots\ldots (5)$$

(In the case of manganese)

$$MnO + C = Mn + CO \ldots\ldots\ldots\ldots (6)$$

The tendency of this carbon monoxide gas is to release itself from the steel by rising to the top of the mould. The bulk of this gas has been produced in that part of the ingot which is last to solidify, as previously mentioned. This part of the ingot is composed primarily

of iron carbides, iron phosphides, manganese sulphides, etc., so that all these elements will be found in excess near the top. Mr. Petinot.

It very often happens that a steel which shows in a ladle test 0.40 carbon will show, when rolled into bars, a variation of carbon between 0.35 and 0.45. If titanium is added, it will prevent, to a great extent, the formation of CO in the moulds, because, as already shown by Equations 3 and 4, the titanium will react on these oxides in the ladle, so that, when the steel is poured into the moulds, the only segregation that will occur is that caused by the chilling action of the moulds.

The speaker's experience has been that a clean product, with very little segregation, can be obtained by the use of titanium, at an additional cost of from 25 to 50 cents per ton of steel treated, and, if this is so, the uniform composition of such a steel will easily warrant small additional expenditure.

LEON S. MOISSEIFF,* M. AM. SOC. C. E.—This paper is most interesting and also most timely. It attempts to find a way to meet the ever increasing demands made on the ingenuity of the engineer and the resources of the capitalist by heavier loads and greater spans. There is no doubt that the author points the right way into the future. There is a need of a material of a higher strength than that of the steel generally used in bridge building to-day. Whether it be nickel steel, as recently used in some of our long-span bridges, or another kind of an alloy steel, still better meeting the requirements of bridge building, there is a technical need of a higher strength steel. Not only would heavy railroad bridges with their increasing freight loads be benefited by the higher steel, but also highway bridges of the longer spans. Mr. Moisseiff.

The advent of heavy automobile trucks has made it imperative to provide heavy floors for bridge roadways. The old rule, of an inch depth of plank for every foot of span for the usual timber floor, died quite some time ago. Nowadays, concentrations of 10 000 to 15 000 lb. have to be taken care of on city highways. Even the old reliable buckle-plate, built in a suspended position, appears to have lost its good character. Recently, it has been found on some highway bridges that the buckle-plates are not sufficiently rigid, and that the pavement fails in consequence. This can readily be explained. The suspended buckle-plates are stiffened by the filling of concrete placed on them. The stiffness thus provided was sufficient for the old-time concentrations from wheel loads, and the pavement was supported rigidly enough to give good results. The increased concentrations of modern traffic, however, overcome the stiffening resistance of the concrete and the buckle-plates, and the curve of the plate is distorted under the advancing action of the load. This results in the crushing and crumbling of the concrete in the trough, and the ultimate failure of the pavement.

To insure the good behavior of a pavement, not only a strong but also a stiff support is necessary. Such support will be furnished by a

* New York City.

Mr. Moisseiff. heavy timber floor with floor stringers at close intervals, or by reinforced concrete slabs. In either case a floor system will be required which will have considerable weight. To take care of this additional weight and load a high-strength steel is wanted.

The author has gone to a great deal of trouble to show the economical advantage of higher steel, and there can remain no doubt as to the desirability of its use for bridges.

Granted, then, the desirability of the higher strength steel, questions of design may well be considered. One of these questions may be: what will be the effect of the higher steel on the stiffness and stresses of bridges built of that material?

The higher elastic limit of the new steel will be utilized, of course, for allowing higher unit stresses, which is the purpose of its use. The coefficient of elasticity being practically constant for kindred steel, the higher unit stresses will result in greater elongations of the individual members and greater deflections of the entire structure. The increased deformation of the trusses means a greater deviation from the original truss form, and will result in much increased secondary stresses.

Assuming, as the author has apparently in the paper, that the allowable unit stresses will retain the same proportion to their corresponding elastic limits as they have in the present practice in the case of carbon steel, the allowable stresses for the hoped for high-alloy steel of 100 000 lb. elastic limit will be nearly three times those of carbon steel with its elastic limit of 35 000 lb. Evidently, the moving load deflections will become considerable and the secondary stresses may attain the rank of primary stresses. How, then, to take care of the secondary stresses in the erection of the bridge, and provide for the deformation of the trusses under moving load, will raise new and serious problems in bridge design.

Another question is how to connect efficiently the high steel members, apart from using pin connections. With common soft steel rivets the splices would be of excessive length, the efficiency of which is subject to much doubt. Consequently, high-steel rivets must be used. Nickel steel members have been designed by the speaker for corresponding nickel steel rivets, but these rivets have not always proved very satisfactory. They are not easily well driven and are difficult to remove. This refers especially to field rivets which have to be driven with a gun or a machine of limited pressure. Now, if nickel steel rivets having an elastic limit of 50 000 lb. present some difficulties, what will rivets of, say, 80 000 lb. elastic limit offer?

Mr. Skinner. F. W. SKINNER,* M. AM. SOC. C. E.—The speaker, having recently had an opportunity of seeing some phenomenally large carbon steel rivets, made for the new Quebec Bridge, would like to confirm to some extent the views of the author and of Mr. Hodge. Some of these

* New York City.

rivets—those for the cantilever arms—were 7 in. or more in length, with a diameter of 1¼ in. Mr. Skinner.

Some experimental work was done in which these rivets were specially designed with a slight taper at the points. They were quenched at the points in cold water. When driven, they were held in place by a pneumatic bucker, and the driving was done with a pneumatic hammer. The pneumatic bucker was a special tool, having a hollow cylinder in which a hammer was arranged. The hammer and cylinder were controlled independently, which enabled the bucker to be held in position, and then when the ordinary hammer was operated on the point of the rivet, the secondary hammer in the bucker was operated, and the rivet was thus driven simultaneously from both ends. The result was that they were exceedingly well driven. A row of rivets was driven partly by yoke machines and partly by the hammer and bucker process, as in the field. When they were cut in two, it was almost impossible to distinguish between them, the field-driven rivets being, for every practical purpose, as good as those driven by machine in the shop. Therefore, it seems quite reasonable to expect that field-driven rivets can be made substantially as good as shop rivets, and, if nickel steel rivets can be driven satisfactorily, it may go a long way toward producing equally good results in the field.

Regarding the possibilities of high alloy steel in bridges, there can be no doubt that the need of it is great and the prospect is alluring. The speaker supposes that it is generally agreed that there is difficulty in securing uniformity in its production. A great many years were required to produce carbon steel which was trustworthy and uniform, and it will no doubt take as long to get alloy steel in uniform condition. The speaker would be very glad to hear from the author about the possibilities of developing a super-strong alloy steel by subjecting finished members to heavy test loads at the shops. It has been pretty well demonstrated that both the elastic limit and ultimate strength of carbon steel, for both tension and compression, can be thus greatly raised. The speaker does not know that this has gone farther than a demonstration, but it was considered more or less seriously for the Quebec Bridge, and perhaps for the St. Louis Municipal Bridge. Such treatment should achieve the double result of increasing the elastic limit and ultimate strength, and therefore the working capacity, and should serve as a valuable test or proof load, disclosing all serious defects.

If an increase of from 20 to 40%—possibly even more—can be practically attained at moderate expense for carbon steel, what would be the result if applied to nickel or other alloy steel? It may be inferred that important results can be obtained. The speaker thinks that it opens up a wide field of conjecture.

Mr. Corthell.

E. L. CORTHELL,* M. AM. SOC. C. E. (by letter).—There is no doubt that the spans of bridges are increasing in length and that there are many locations where the kinds of steel described and treated in this paper would be used to bridge over wide expanses of water not now possible, if they could be manufactured at a reasonable price.

All engineers who are contemplating such structures will take great interest in the author's suggestions. It is to be hoped that the funds necessary for the suggested series of experiments may be found. The results would be of use, not only in the United States, but elsewhere, such as, for instance, in the proposed cantilever bridge over Sidney Harbor, Australia, the main span of which is to be 1 600 ft. long and 94 ft. wide between centers of posts, for four railway tracks, main roadway, motor roadway, and footway. Mr. Bradfield, the engineer, is soon to be in America, and this paper will interest him.

The extra price of 2 cents per lb. asked by manufacturers for the proposed high-alloy steel would restrict its use, but if it was called for by engineers, in sufficient quantities to warrant its general manufacture, no doubt the price would be cut in two, as the author indicates.

The author's remarks on page 18 about live loads are quite important. He states that:

"The live-load stresses on the main truss members will never be quite as great as they are computed, because, first, the trains on the two tracks never advance together so as to produce maximum web stresses; second, such trains are not likely ever to cover entirely the bridge or even any individual part of it, except, perhaps, the central span; and, third, it is improbable that any load of cars, unless they be ore or coal cars, will ever be uniformly full and loaded to the assumed limit."

All of which is true and might well be taken into account in the actual design of long-span railroad bridges.

It is customary at present to allow for a train load of 5 000 lb. per lin. ft. (from 3 000 to 3 500 lb. per lin. ft., 20 years ago). Not many cars weigh (loaded) 5 000 lb. per lin. ft., and these are not often loaded to their full capacity. A car having a capacity of 100 000 lb. will weigh about 50 000 lb., and is 40 ft. long. This brings the load to 3 750 lb. per lin. ft., which is not much more than that allowed 20 years ago. Are designing engineers expecting still larger cars than at present? It would seem almost impossible to increase the loads per linear foot to more than those which now obtain.

Certainly safety should be the desideratum, but if the dead load of a bridge can be decreased materially by using judgment in the application of live loads in designing, a great deal of money would be saved, so much, in fact, that cases might arise where the difference in cost would make practicable a project which otherwise might be impracticable.

* New York City.

This, and substituting for carbon steel a steel of a "high, cheap alloy, suitable in every particular for bridges", which, the author says, "is now before metallurgists and the builders of large metallic structures," may reduce weights and consequent costs to such an extent as to be of immense advantage in the projects of railway bridges of long spans. Mr. Corthell.

The author is right in his statement that "the onus is on the Engineering Profession to see that the necessary experiments are arranged for." It is to be hoped that the paper will receive the discussion that its importance and character deserve.

W. H. Warren,* M. Am. Soc. C. E. (by letter).—This paper is of unusual interest to the writer, as it is proposed to build a cantilever bridge over Sydney Harbor of 1 600 ft. span. In giving evidence before the Public Works Committee of New South Wales, in May, 1912, he strongly advocated the use of nickel steel in the cantilevers and suspended girders, with carbon steel in the floor-beams and stringers, and also in the systems of bracing. He also directed attention to Mr. Waddell's paper† on "Nickel Steel for Bridges," as supporting his own argument that considerable economy would result in the use of nickel steel on account of its higher elastic limit, and the fact that it can be worked under all the necessary manipulations in the shop in a satisfactory manner, and also that its other physical properties are at least as good as carbon structural steel. Mr. Warren.

If an alloy steel could be produced having an elastic limit of 70 000 lb. per sq. in., with its other necessary qualities suitable for bridge work, at a moderate cost, a great advance would at once become possible in the art of the bridge builder, so far as the design of long spans is concerned. The paper shows that such a material may be reasonably anticipated in the future, provided sufficient inducements are given to the metallurgist and the steel manufacturer, and the greater economy that would result in the use of steel having a still higher elastic limit is clearly indicated in the diagrams which have been worked out in such a painstaking and elaborate manner. The paper is most opportune, and it suggests and indicates the probable method of procedure in a line of research which, it is hoped, the metallurgist will seriously consider. The writer is of the opinion that members of the Engineering Profession all over the world owe a debt to Mr. Waddell for the notable papers which he has contributed on the design of bridges.

David A. Molitor,‡ M. Am. Soc. C. E. (by letter).—This most interesting paper represents many years of labor and experience, and, as a contribution to engineering knowledge, should be accepted with due appreciation and regard. Mr. Molitor.

* Sydney, New South Wales, Australia.

† *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101.

‡ Toronto, Ont., Canada.

Mr. Molitor. The author's conclusions relative to the advantages to be derived from the use of "high-alloy" steels in bridges, both in point of economy and greater span lengths attainable, are true beyond question, and, as pointed out in the paper, the problem of finding a suitable cheap alloy steel is now before the metallurgists and the builders of large metallic bridges.

It might be said that this problem has always been before the manufacturers, and possibly also the metallurgists, at least since the construction of the Eads Bridge at St. Louis; but, what has been accomplished, and who is most interested in progress along these lines?

Aside from a few innovations, chiefly with the aid of alloys such as nickel, aluminum, chromium and manganese, to raise the elastic limit and ultimate strength, and titanium as a purifying agent, little has been accomplished to improve the quality of steel for bridge and structural uses since the introduction of the open-hearth process. However, the two great defects in all manufactured steel are segregation and piping, for which no remedy is known to mill operators except the costly and wasteful method of discarding from 15 to 40% of the ingot, and, further, scrapping all material showing defects in the finished product. If these defects were readily discernible, this method might insure good results, but at high cost.

Both segregation and piping are local defects; the former is always hidden in the finished material, and the latter is usually invisible except when it appears as a noticeable surface defect. Hence, it follows that no specifications can be written so as to insure a high-grade homogeneous material, in spite of the most painstaking mill inspection.

Engineers prescribe both the chemical purity and physical requirements for bridge and structural steel, and require that a test shall be made from a specimen taken from each ingot, and on the results of these tests the metal is either accepted or rejected, reserving the right to reject such of the accepted material as may develop defects in course of manufacture. The presumption is that all material not rejected in the mill or at the shops is uniformly good and up to the specifications.

However, nothing could be more erroneous, and, even if every single piece of rolled material were tested in like manner, this could not be accepted as absolute proof of quality. For there might be a hidden flaw or pipe, or spot of segregated metalloids, anywhere in the finished piece, which would never be revealed by any test, but might ultimately come to light through failure.

A careful perusal of the Report of the Committee on Rails, of the American Railway Engineering and Maintenance of Way Association,* will convince the most skeptical of the truth of what has just been stated.

Although segregation and piping defects are likely to occur in any or all material, they are less likely to produce disastrous results in

* Vol. XII. Part 2, 1911.

bridges than in rails, owing to the more strenuous service of the latter. It is for this reason that ingots for rail steel are cropped about 30%, on the average, and bridge steel is cropped about 20%, on the average, with a minimum of about 12 per cent. Mr. Molitor.

It should be noted that Bessemer steel is less subject to piping than is open-hearth steel, and that the latter suffers less segregation than the former, owing to its lesser sulphur and phosphorus contents and lesser quantity of metalloids.

Although these conditions exist in bridge steel of the present day, their deleterious effects are largely overcome by using moderately low working stresses. It is well, however, to reflect on the conditions which would inevitably follow when the same basic material is raised to higher strength by using an alloy or higher carbon contents. Unless segregation and piping can be effectually eradicated, it would seem quite out of the question to consider high-alloy steels safe for large bridges, as defects of this class would become far more serious in slender sections with high unit stresses. In other words, a condition which might be tolerated in medium steel, might become prohibitive in the high-alloy steel.

The crucial point of this discussion rests on the willingness and on the ability of the steel manufacturer, first, to improve the quality of the present-day product by the prevention of segregation and internal pipes, and second, to raise the elastic limit and ultimate strength of this homogeneous steel by the use of alloys, taking care to secure proper ductility in the high steel. Carbon might be used to a certain extent, but, in all probability, at a considerable sacrifice of ductility.

From the manufacturer's viewpoint, however, innovations of this nature do not interest him so long as his experience is profitable and he can dispose of his entire output without difficulty. The operating department of a steel plant aims merely to produce tonnage, and the commercial department seeks to dispose of this tonnage to the best advantage.

Albert Lucius, M. Am. Soc. C. E., at the close of his discussion says:

"The engineer must get next to the metallurgist. It is highly probable that any metallurgist who knows how to produce a better steel alloy at commercial value will succeed very promptly in getting it on the market, * * *."

This is quite an erroneous view, for it rests entirely with the operating manager of a steel plant whether or not he will interest himself in what the metallurgist has to offer, and the metallurgist, in turn, is not getting things on the market. In other words, the metallurgist, who is appealed to as the savior in this matter, is quite helpless to do anything further than possibly say he knows how the task might be accomplished.

In all the learned discussions precipitated by investigations of rail failures, the conclusions reached, by metallurgists and others possessing

Mr. Molitor. a wide experience in testing and inspecting steel, are that practically all the failures are due to piping and segregation. However, the only remedy proposed by our learned friends, to overcome these serious defects, is to continue cropping off the ingots until the remaining portions appear to be uniformly good and sound. These same experts when advised that the difficulty could be overcome successfully by a very simple and cheap process which would be communicated to them if they were interested, failed to go further than to acknowledge the receipt of the writer's letter of more than a year ago.

This is a fair sample of the interest manifested by those who are most in the public eye.

It may add to this discussion to enter somewhat into detail as to the phenomena of piping and segregation, as comparatively little is known with certainty, and, hitherto, all attempts to overcome these defects in steel have proved only partly successful.

Although chemical reactions of a most complex nature are known to take place between the various impurities, sulphur, phosphorus, silicon, and oxygen, with iron, manganese, and carbon, forming compounds which are known as metalloids, it is practically impossible to show the exact nature and composition of these metalloids by chemical analyses. It is also quite probable that the metalloids in the molten state are very different from those in the frozen condition. There is also a considerable gas formation in the interior of the molten metal, chiefly caused by the oxidation of the carbon and some of the impurities.

It is also known that the metalloids have a lower specific gravity than iron, and that their melting points are lower than that of iron, which being the case, the metalloids will cool more slowly than iron, and will naturally free themselves, to a certain extent, and on account of the difference in specific gravity will be forced laterally and upward into the lake of molten metal and continue in this action, so that the last quantity of metal that is cooled will contain a concentration of these segregated elements. The gases are similarly displaced, and, being extremely light, will hasten the collection of the metalloids into nodules which will travel upward along lines of least resistance as the metal cools in the mould. It is this effect, due to the difference in melting points, and the gravity displacement of the metalloids and gases, which gives rise to segregation and may, therefore, be classed as a physical phenomenon.

Piping is also a physical phenomenon, being caused by the shrinkage of the metal in the mould as it cools, and is the result of the following conditions: The metal is poured into the cold mould, and, as it strikes it, it chills or freezes to it at the bottom and sides, the chill being continued to the top level of the poured metal. The cooling or freezing takes place successively toward the top and center of the mould, and as the metal cools it naturally contracts and draws its supply for contraction from the molten lake in the center which sinks as the

demand is made on it by the contraction of the surrounding metal, the result being the formation of the funnel-shaped cavity which frequently reaches below the center of the mould. This cavity may contain, in some instances, hanging walls or bridges, which may seal across the lower part of the pipe, resulting in a hidden or concealed pipe. This may not be detected, either in the bloom or in the finished product. Mr. Molitor.

Segregation and piping are thus seen to be physical manifestations which cannot be prevented by any chemical process or treatment.

Although a chemically pure steel would necessarily be free from segregation, yet no steel, or, in fact, no metal, can be poured into an ingot mould without showing a pipe. The great necessity of freeing steel from internal pipes is thus made apparent.

The use of titanium unquestionably improves steel, and especially when added to Bessemer steel, which is high in sulphur and phosphorus, and also in gases. For this reason, Bessemer steel boils more in the mould. The addition of titanium is especially useful to quiet the metal by preventing the gas production through combinations with nitrogen and oxygen, forming light substances which rapidly rise to the surface of the ladle, purifying the steel at the expense of the titanium, very little of which remains in the steel. Hence, titanium somewhat reduces segregation, but does not affect piping.

More than a year ago the writer's attention was drawn to a process of casting ingots, by which the gases are all expelled, the metalloids are prevented from rising, and the steel cools gradually from the bottom upward instead of inward from the sides of the mould, thus preventing unequal contraction along different elements vertically in the mould. This is all accomplished by one operation, without loss of time in casting ingots, and at little or no expense outside of the apparatus on which the mould is placed while making the ingot casting. The process is one of forging the liquid steel by imparting vertical vibration to the mould and its contents, and is called by the inventors, Messrs. Maxwell and Lash, "The Liquid Forged Steel Process."

From the foregoing description, and the phenomena attending ingot casting, it is quite clear that segregation and piping can be completely obviated, thus making it possible to produce a perfectly uniform and homogeneous steel in which the ultimate strength and elastic limit are raised about 20% by the de-gasifying effect of the process.

The loss due to necessary discard of the ingots for bridge steel is reduced from about 20% to 3 or 4%, and, for rail steel, the reduction is from an average of 30% also to 3 or 4 per cent. This would result in an actual saving of from $1.50 to $2.00 per ton for steel production; the cost is merely an interest charge on the value of the machinery used and a small royalty on the patents, to say nothing of the far greater value of the finished product and the savings brought about by lesser rejections of the rolled steel.

Mr. Molitor.

That these statements are correct cannot be disputed when the tests which were made last year by the patentees are examined. Hence, the introduction of this process into the large steel mills would be somewhat of a revolution in steel manufacture as well as in the vastly improved quality of the finished product.

In the present grades of steel, defects due to segregation and internal or concealed pipes represent losses to the customer, and the visible pipe is a direct loss to the mill through discard. Hence, it is, to both customer and mill, that the introduction of this new process commends itself, and both parties should be equally interested in seeing it introduced.

However, it is more than likely that the demand for the better steel must be first created before any mill will undertake the production of such an article.

To the writer's mind, there is no question regarding the possibility of producing high-alloy steels, of the kind sought by modern bridge builders, whenever it suits the convenience of the mill manager to disabuse his mind of several preconceived views and to express his willingness "to be shown". The writer also ventures to add that aluminum, possibly with nickel, will offer the most desirable combination with steel, owing to the great ductility it imparts.

Returning to questions of design, it should be observed that our present methods would suffer numerous modifications when dealing with high-alloy steels, even in small bridges, as our knowledge of secondary and impact stresses would require. Deflections due to moving loads would be increased owing to the higher unit stresses used and the practically constant extensibility of all grades of steel. Secondary stresses might not suffer much increase on this account, because the connections between slender high-steel members would be less rigid, though impact stresses might be increased because the ratio of live load to dead load would be increased along with the greater flexibility of the structure.

As discussions of this kind are sometimes fruitful in bringing about advancements in engineering science, the writer expresses his high appreciation of the author's efforts in the direction of progress, and closes with the optimistic feeling that some substantial gain in the art and science of bridge building may soon come as a reward, provided the steel makers will do their part.

Sir B. Leslie.

Sir Bradford Leslie* (by letter).—In India the river beds are generally alluvial for an unknown depth, and piers of bridges having to be sunk to a great depth are very costly, consequently, it is economical to use spans of considerable length, but the writer is not aware of any cases in which exceedingly long spans are likely to be required. Nevertheless, the paper is of the greatest interest to bridge engineers.

The author has fully established the necessity for experimental investigation of the problem of discovering a high-alloy steel which

* London, England.

can be subjected to heating and working at temperatures attainable in the ordinary operations of bridge construction, without impairing its strength, and which can be placed on the market at a price moderately exceeding that of the ordinary carbon steel of commerce. Such a metal would greatly reduce the weight and cost of girder bridges of moderate spans. The percentage of nickel required is so small that it seems probable the ores will be discovered in sufficient abundance, and, if so, the demand for the material must soon result in the practical realization of the author's far-sighted ideas on bridge construction. Sir B. Leslie.

The writer belongs to the iron age, which is ancient history, but can well remember the relief afforded to bridge designers by the increase of 50% in the elastic limit when carbon steel superseded iron as the material for bridges, and he hopes he may live to see the further improvement that will result from the general use of high-alloy steel for that purpose.

Although the writer feels that he is not qualified to offer useful criticism of the investigations described in the paper, he desires to state that his experience ranks him among the unbelievers in the dangers of so-called "impact". If that bugbear can be dismissed, it will effect great economy in girder construction. He is equally skeptical as to the supposed injurious effects of alternating stresses which do not exceed the elastic limit.

A train does not drop on to a bridge, it rolls on and off; at whatever speed it may be traveling, the pressure is applied and relieved gradually—a mass falling by gravity moves but slowly when it begins to fall—any blow there may be is due to the action of the springs. Velocity in its application does not affect the stresses distributed over a given area of material by any load. At 60 miles an hour the axle of a vehicle may be subjected to alternating stresses at the rate of 10 per sec. but not to "impact"; properly lubricated, the axle keeps cool. The worst "impact" that axles get is at worn rail joints which strike a blow felt through springs and cushions, and is the principal cause of axle failure. "Impact" is locally and not otherwise destructive. If a bar of iron, such as a 6-ft. length of rail, be suspended by one end and constantly struck four or five blows per sec. by a hammer, heat will be evolved by the superficial destruction of the metal; such destruction will be local and cumulative; the entire bar will vibrate; but, however long continued, the vibration will not affect the properties of the metal. In a similar manner, the top of a pile is punished by hard driving and the remainder of the pile suffers no injury, though, it is true, the pile receives but a few hundred blows.

If bridges suffer from "impact", it must be transmitted through the rails. Rails are worn out by attrition; excepting at worn joints, there is no "impact"; millions of loaded axles rolling over rails at high velocity do not impair the strength of the metal.

Sir B. Leslie.

Many years ago the writer made careful experiments on scrap rails before using them for structural purposes*. The rails were of iron, of the old double-headed section formerly in use in England. Pieces were cut from both heads and tested under tensile and compressive strain, also for ultimate extension and contraction of area. In no instance was it found that the scrap metal had lost any of its original properties, under millions of tons of traffic resulting, not in "impact", but in constantly alternating stresses. "Impact", indeed, there is, at the joints, causing failure of the ends of the rails.

Prior to the last 25 years, no attention had been given to "impact", and no account had been taken of it in designing girders. The writer has never met with any authentic instance of failure on that account. On the contrary, many large bridges, both of iron and steel, are now carrying axle loads in excess of what they were originally intended to carry, without any indication of failure, except in the platform system, where increased panel loading has, in some cases, involved strengthening.

The only precaution advocated by the writer to mitigate "impact" (the effects of which in short spans are injurious to the masonry abutments, rather than to the girders themselves), is to build bridges up to, say, 100 ft. span to carry the ordinary track laid in good ballast. This facilitates maintenance, prevents the passage of the bridge being felt on board the train, and the shaking of the masonry abutments, and saves the necessity for allowance for supposed "impact". Such saving is discounted by the girders having to be provided with flooring, and built strong enough to carry ballast. However, there can be no comparison in the comfort of traveling over a ballasted bridge carrying the ordinary track, and running over a rattling, noisy structure in which the track is fixed directly to the girders.

The writer infers, from the paper, that it is customary to build long-span bridges for double track, presumably because the increased width between the girders relieves the bending moments due to wind pressure, and improves the lateral strength of the structure generally.

If worked on a proper system, a single-line bridge will accommodate a very heavy traffic. Two trains following each other every 10 min. gives twelve trains in each direction per hour. Where intensity of traffic absolutely demands it, the expense of a double-line bridge must be met.

Instances have occurred on the 5 ft. 6-in. gauge railways in Bengal of trains being brought to a standstill and overturned by wind pressure. This indicates a side pressure approaching 40 lb. per sq. ft. over considerable areas. No such accident has hitherto occurred on a bridge

* Owing to heavy rates of freight from India to Europe, scrap material which could not be rerolled or worked up in India, in about 1878, was practically unsaleable, consequently its use for structural purposes, instead of importing new material, was very economical.

in India, though it is probably what took place at the collapse of the Tay Bridge in 1869, and it is necessary, therefore, to take account of the possibility. Sir B. Leslie.

It is suggested that all important bridges should be equipped with a reliable anemometer interlocked with the signals, so that whenever the wind attains a velocity exceeding, say, 80 miles per hour, or 17 lb. per sq. ft., the traffic over the bridge would be stopped; considering the risk, this is only a reasonable precaution. Thus, in the case of wind pressures exceeding 17 lb. per sq. ft., the effect on the bridge would be limited to that on the structure only; below that it would be necessary to take account of the wind pressure on the train as well as on the bridge itself. Such an arrangement would enable the economy of single-line bridges to be realized wherever practicable and desirable.

Doubtless, however, all these points have received due consideration in the preparation of the paper. They are all the observations that the study of this valuable paper, considered in the light of 60 years' experience in bridge building, enables the writer to offer.

JOHN C. FERGUSSON,* ESQ. (by letter).—The amount of time and energy the author has devoted to his subject, and to the preparation of his valuable diagrams, showing, by comparison, the advantages of using steel of higher tensile strength in long-span bridges, shuts out all but the most friendly criticism. Yet facts are stubborn things; we do not look at them from the same plane, so we cannot all see alike. It is this very difference of view which, in the long run, gives the real value to the exchange of ideas among civil engineers, by bringing things into true focus. Mr. Fergusson.

The use of nickel steel cannot be adopted, generally, as the best material for all classes of bridgework; its particular advantages adapt it more for use in bridges of very long span than in short ones. Its special advantages—elastic limit and tensile strength, hence lightness—over the best classes of open-hearth carbon steels, are only obtained in nickel steels of the very highest grades; and their cost would be prohibitive in ordinary structures. Carbon steels, for a tensile strength of 40 tons per sq. in., have advantages over nickel steels carrying low percentages of nickel. The open-hearth carbon steel is a more reliable product, it is stronger in compression, has greater resistance to impact, its manufacture and testing are extremely simple, and the main results are constant; whereas, the testing of nickel steel during the process of manufacture is very complicated, the material is subject to great variations; and, further, the combinations of carbon, manganese, titanium, and tungsten, even in the smallest fractional percentages, produce most unlike results.

It has been found that the addition of nickel to iron causes the nickel to act as a flux on the carbon, manganese, and other minerals in combination with the iron; and the anomalous results produced

* Birmingham, England.

Mr. Fergusson. vary very much more owing to slight changes in the proportion of carbon, manganese, or other metals in the chemical composition of the ingot, than in the mere change of proportion in the parts of the nickel to iron.

The idea of adopting a standard specification for nickel steel, to be used in bridgework, seems to be wrong, because the conditions required for various spans and loads are so variable. Would it not be wiser for a civil engineer to state his requirements and see that they are carried out, leaving the actual manufacture of the metal to the metallurgists and ironmasters?

To use a nickel steel of very high tensile strength, chiefly to save weight in the bridge material—unless where the conditions are imperative—would seem to be extravagant, and even unwise. There must be risk in trusting to the highest limit of tensile strength of a manufactured steel which is known to be very variable. The mere saving of weight in bridges of moderate span would hardly warrant the risk, coupled with the extra cost of material of higher grade. Standard specifications for nickel steel would be worthless unless they met the highest conditions and requirements for the longest bridges mentioned by the author. It is for bridges of this class, of exceptional length, that he hopes to find a sufficiently strong and suitable material in nickel steel.

The author, in his previous paper,* states that the best percentage of nickel was found to be 3.5. The allowance of impurities in the nickel steel was put as phosphorus, 0.03%; sulphur, 0.04%; silicon, 0.04%, and manganese, 0.75 to 0.85%, which produced an elastic limit of 61 300 lb. and an ultimate strength of 99 300 lb. per sq. in. The resistance to impact was less than that of carbon steel, the relative values being 87 and 73 to 100, for low and high nickel steel.

The author also states that the nickel present in larger quantities makes the material unworkable in the shops.

If the author is still of these opinions, and if we may regard his present able paper as supplementary to his former one, the foregoing statements deserve consideration, because they do not agree with the general idea as to the chemical composition of a very high-class nickel steel that would best fill the requirements of the extraordinarily long bridges he has mentioned. The proportion of 3.5% of nickel, which he gives, appears to be too low. As he does not mention carbon in his analysis, it may be presumed that he has eliminated it; surely this is a difficult operation in commercial manufacture? This is a most important point, for, with a low percentage of nickel, the reduction of carbon below 0.20 becomes essential.

* "Nickel Steel for Bridges", *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101. Abstract in *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVI, p. 345.

Provided the carbon is kept low, there is a constant rise in the tensile strength of nickel steel, in proportion to the increase of the nickel present, up to 12 or 15 per cent. The increase of nickel requires a decrease of carbon, together with an increase of manganese, otherwise, although the breaking load may be the same, the value of the elastic limit will fall rapidly. Mr. Fergusson.

The writer has pleasure in acknowledging that some of the earliest and most successful workers of nickel steel are Americans. The Bethlehem Iron and Steel Company, the Homestead Works, of Pittsburgh, and others, are well known.

There are a few points about which there appears to be considerable difficulty to agree. The author states that steel containing more than 3.5% of nickel was unworkable, yet in Sheffield they found no difficulty in working material containing nickel, 7.65%; manganese, 0.68%; and carbon, 0.17%, although it was hard.

In America there is a preference for adding nickel in the form of iron ores, whereas in England it is preferred to add the nickel pure. In Sheffield it is found difficult to produce forgeable bars in the absence of manganese, and it appears to be impossible to get satisfactory results.

The writer ventures to point out two disadvantages in nickel steel for bridgework:

(*a*) The difficulty of welding, and working;
(*b*) The necessity of keeping the carbon extremely low—under 0.20%—whereas the fatigue resistance of steel is increased enormously by raising the percentage of carbon.

Notwithstanding any difference of opinion about the composition of the most suitable steel for bridge construction—on which discussion was invited—the writer joins heartily in acknowledging the author's efforts, and particularly the value of his diagrams, as a direct advance in bridge construction.

V. E. DE B. DE BROË,* ESQ. (by letter).—The writer has read this paper with interest, and begs to offer the following remarks. Mr. de Broë.

Spans.—As compared with Indian practice, the length of the spans required in America is remarkable. The Government of India railway bridge regulation tables are worked out for spans of from 5 to 600 ft., and usually, the engineers can negotiate big sandy rivers on deep wells with girders of from 150 to 250 ft. span. The writer believes that the only railway bridge in India of greater span than 600 ft. is that over the Indus at Sukkur, which is between 700 and 800 ft. Of small spans, ranging from 10 to 100 ft., Indian railways have a great number, and additional ones are being erected daily. It is readily understood, therefore, that, with engineers in India, the use of a metal of excessively high tensile strength is not of so much con-

* London, England.

Mr. de Broë. sequence as it is with American engineers, faced as they are with the problem of bridging vast clefts and chasms with single openings.

Strength of Metal.—In preparing specifications, the writer works to the "British Standard Specification for Structural Steel" (revised in August, 1912), which enjoins an ultimate tensile breaking strength for ordinary girder sections of "between the limits of 28 and 33 tons per square inch." As the elastic limit is generally not less than 50% of the ultimate strength, and good firms usually work to this without extra charge, it follows that

$$\frac{\overset{\text{Tons}}{33} \times \overset{\text{Pounds}}{2\,240}}{2} = 36\,960 \text{ lb.}$$

is the extreme elastic limit per square inch reached in practice. The author states that ordinary carbon steel is usually furnished by American manufacturers with an elastic limit of 35 000 lb., or an ultimate tensile strength of approximately 32 tons per sq. in., so that in this respect, Indian engineers are on an equal footing, and it is not until American engineers begin to mix an alloy that they get a higher elastic limit.

It is noted that the author was successful, with French manufacturers, in realizing the high standard of 45 kg. per sq. mm., equivalent to 28½ tons per sq. in., or an ultimate breaking strength of 57 tons per sq. in. Nothing of this sort has ever been heard of in Indian practice. The author also expresses himself as sanguine of reaching in America 60 000 lb. = 27 tons elastic limit, or 54 tons ultimate strength; and his researches have led him to believe that even the colossal figure of 100 000 lb. = 44 tons elastic limit, and 88 tons ultimate tensile strength, is realizable by persistent and enduring experiment until the right mixture has been obtained. The writer has not had the pleasure of reading the author's former paper, "Nickel Steel for Bridges," and is not chemist or metallurgist enough to form an opinion as to what can be done in the laboratory or in small melts in the production of an alloy greatly superior in strength to carbon steel; it is obvious, however, that should an alloy ever be produced approximating in strength three times that of steel (at a commercial market value), it is not only America that will benefit, but every railway in the world will be in a position to revolutionize its bridge stress and strain calculations with a view to economizing in weight of metal.

The estimated cost of the author's proposed experiments, \$100 000 = £20 000, or even twice this sum, as suggested, is a very large one for private enterprise, but not for a Government or a group of Governments to contribute, and it is further to be noted that, should the experiments result in the success which the author anticipates, the use of the new and stronger metal could and would be extended to

other forms of construction besides bridge girders. There are no millionaire philanthropists in India, and so far as that country is concerned, anything of the sort suggested would have to be done by the State. Mr. de Broë.

Cost of Steel.—On page 21 it is noted that the author is handicapped with the high cost of 4 cents per lb. for steel for plate-girder spans, or £18 sterling per ton. The highest price paid in England (for export to India) during the last few years, was £13/10/0 per ton, and, even adding sea freight, it would seem that, for ironwork required reasonably near the seaboard (and not involving long railway carriage to the site of the works), London would afford a cheaper market than America.

Impact.—Impact is a subject to which the writer has devoted a great deal of study of late years, and he has come to the conclusion that it may be totally ignored, provided a working stress not exceeding 8 tons per sq. in. for steel is allowed, and a factor of safety of 4 is used.

Of course, there is much diversity of opinion on this subject in the Profession. The Government of India enjoins the use of the Pencoyd formula:

$$I = \frac{300}{L + 300} = \text{a coefficient by which the live load is to be multiplied;}$$

where L = length of loaded distance, in feet, producing maximum strain.

This gives:

In 5-ft. spans	98%	of moving load,
In 600-ft. "	33%	"
As against	78%	as calculated by the author's formula,
and	36%	

$$I = \frac{40\,000}{L + 500}.$$

The writer has recently come into possession of the Swiss Government revised regulations (issued in 1913), in which the formula for impact is $2(15 - l)$ per cent. of the moving load, l being the span, in meters.

It is obvious from this that no impact allowance is enforced for spans of more than 50 ft., and the percentage works out: For 5-ft. spans 27% only, as against that of the Government of India, 98%, and the author's 78 per cent.

The Swiss working stress is 6.98 tons per sq. in. for steel.

The Board of Trade (England) fixes a working stress of $6\frac{1}{2}$ tons for steel, which is intended to cover impact allowance. Mr. A. H. Shield, writes as follows:*

"The limiting stress of 5 tons per sq. in. for which the early bridges were designed did, in fact, provide for impact, and was

* *Railway Gazette*, June 12th, 1914.

Mr. de Broë.

fixed by the Board of Trade in 1859, some years after experiments had been made by a Royal Commission, appointed to consider the application of iron to railway structures.

"In the course of these experiments the additional strain upon bridges due to a rapidly moving load was measured by the increased deflection. A fixed permissible stress obviously failed to graduate the impact according to the span, and the allowance made in fixing 5 tons was probably insufficient for spans under 20 ft., for which at that time the use of wrought-iron was, however, uncommon. The point which it is important to bear in mind, is that the old standard admissible stresses, 5 tons for wrought-iron and 6½ tons for steel, did include an average allowance for impact, and that this should be taken into account in making a more accurate provision according to the span, or more properly, according to the shortness of the period during which the load comes upon the bridge or part of the bridge under consideration. It is only if this is neglected that the application of a graduated allowance for impact leads to a general increase in the weight of bridges.

"The sufficiency under average conditions of a factor of safety of four without further allowance for impact, is substantiated by experience in this country for 55 years; and any rules which require a greater allowance under similar conditions are indicative either of a desire to provide indirectly for the growth of the load to which structures may be subjected, or to provide a wider margin against errors of design and workmanship than has proved to be necessary under the conditions of responsibility and supervision that have obtained in the past.

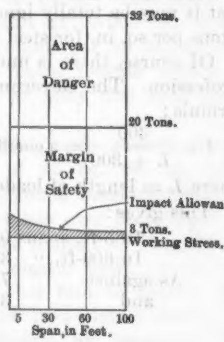

FIG. 27.

"Whichever may be the motive of the authority responsible for the increased burden it is equally deserving of the attention of those interested financially in the promotion and construction of railways in our colonies and dependencies."

The writer has not space or time to state exhaustively his reasons for recommending the total exclusion of the impact factor in bridge calculations, and perhaps the author would not thank him were he to enlarge on the subject, for a full exposition of his views would cover many sheets of paper. Briefly, however, he may say, first, that he does not think deflection tests (on which many of these formulas are based) are reliable. The following is from the Government Inspector's report on two spans of 40-ft. girders which the writer's company recently erected in Southern India:

"Two spans were tested with a B-class engine, one a dead load test and the other at 45 miles an hour. The deflection in both cases was 0.15 inch, which is very satisfactory."

This is one of the many instances. Mr. de Broš.

In the second place, if a diagram is plotted, in the shape of a column (Fig. 27), and divided into three lengths represented by:

(1) A line across at, say, 8 tons (below which the girder may be loaded);

(2) A line across at 20 tons (below which is obtained an area of safety, which may be encroached on occasionally); and

(3) A line across at 32 tons (below which is represented the danger zone, never to be encroached on);

we may see how very small is the margin of risk if, above the 8-ton line, we hatch in the professional allowance for impact. We may also see how large is the margin of safety and how remote the area of danger.

In the third place, the various working stresses allowed by different authorities, and the factor of safety of four, are in themselves wholly empirical, and the writer sees no logical sequence in building, by the application of quasi-scientific formulas, on an empirical foundation. Nevertheless, he would be very interested to see the "more accurate formula" which the author has evolved from deflection experiments, mentioned on page 7 and the data on which it is based.

M. Séjourné, Esq.* (by letter).—The writer takes great interest in the problem of the use of steels of high elastic limit for the construction of bridges, and would second the author's efforts to obtain in France—at least for large works—the use of a different kind of steel from that which is now specified, and having the following characteristics:† Mr. Séjourné.

$R = 42$ kg. per sq. mm.

$E = 24$ kg. per sq. mm.

$A = 25\%$ in a standard length, $L = \sqrt{66.67\ S}$.

In France it is not usual to construct such great bridges as in America; consequently, the use of steels of high resistance is not as necessary, and the French steel works hesitate to supply themselves with tools for producing in great quantities steels which they are not sure of selling.

The writer is at present engaged in designing a large metal bridge over the Durance, the longest span being 121 m. (397 ft.). On account of the importance of this work, one of the largest steel works in France was requested to quote prices for furnishing carbon steel having the following characteristics:

$R > 50$ kg. $E > 27$ kg. $A = 20$ per cent.

* Ingénieur en Chef du Service, Chemins de Fer de Paris à Lyon et à la Mediterranée, Paris, France.

† It will be noted that these characteristics differ from those given to the author and reproduced by him in his article in *Le Génie Civil*, No. 15, August 7th, 1909.

Mr. Séjourné.

The steel works replied that it would agree to these specifications, provided that an increase in price of 1 Franc per 100 kg. more than for ordinary 42-kg. steel was allowed, but without impact test.

In using steels of high elastic limit it seems to the writer to be indispensable to have all the necessary guaranties with reference to its brittleness, and to have recourse, for that, to methods even more stringent than those now used for 42-kg. steel.*

Although French steel works allow tests for brittleness for high-priced special steels, they are generally opposed to them for steels of ordinary make designed for bridges and metal frameworks. These conditions will perhaps be bettered, thanks to developments in the manufacture of steel due to the electric furnace, which produces a material that is not brittle because it is pure. Unfortunately, its price is still too high.

The writer commends the author's endeavors to discover a constituent alloy of great tensile strength, high elastic limit, and of reasonable price. The problem is doubtless difficult to solve, but it is not beyond the science and ingenuity of American engineers, and it is hoped that the author's researches and tests may result in the desired metal.

Mr. Norris.

GEORGE L. NORRIS, ESQ.† (by letter).—The author has shown the great advantages to be derived in bridge construction, especially for long spans, by the use of steel with high elastic limit. To obtain the high elastic limits which he mentions—60 000 to 100 000 lb. per sq. in.—it will be necessary to use alloy steels.

With alloy steel it is entirely feasible to obtain elastic limits of from 90 000 to 100 000 lb. by heat treatment, and even also in the condition as rolled, but this, of necessity, would increase considerably the difficulties of shop manipulation. In the condition as rolled, the alloy steels would naturally be too variable in hardness for safe use.

In the case of nickel steel, in order to attain an elastic limit of 60 000 lb. or more, it is necessary to increase the percentage of carbon to undesirable limits.

To produce steel having an elastic limit approaching 100 000 lb., and workable under ordinary shop manipulations, the writer believes that recourse will be had to the use of vanadium alone, or with some other metal such as chromium in the steel. Vanadium is undoubtedly the element which, together with carbon, acts with the greatest intensity in improving alloys of iron, that is, in very small percentages. The quantity of vanadium generally alloyed with steel, excepting tool steels, is about 0.15 or 0.20 per cent. The price of vanadium has been

* In reference to the information given to the author, and reproduced by him in his article in *Le Génie Civil* (August 7th, 1909), it is said that the specifications of the French railroads "do not require impact tests, which is greatly to be regretted." This is true, but the writer does not make use of these specifications in his office, and for 12 years he has required such tests on all rolled steel.

† Metallurgical Engr., American Vanadium Co., Pittsburgh, Pa.

reduced 60% within the past 3 years, and it is now low enough to bring it within consideration for use in steel for eye-bars and other parts of long-span bridges. Mr. Norris.

Vanadium has generally been used in combination with chromium, although when used with nickel or nickel and chromium it gives greatly increased physical properties; these latter combinations are naturally more expensive. Added to simple carbon steel, especially if the percentage of manganese is more than 0.60%, it gives an increase of about 40% in the elastic limit and about 20% in the tensile strength, with practically the same or even greater ductility.

What is known as Type "A" chrome vanadium steel will fulfill the requirements of high elastic limit and be workable under shop manipulations. The cost of this steel would be only slightly in excess of that of 3½% nickel steel. Its chemical range would be:

Carbon	0.17 to 0.27	per cent.
Manganese	0.40 to 0.60	" "
Chromium	0.60 to 0.90	" "
Phosphorus, not more than	0.05	" "
Sulphur, not more than	0.05	" "
Vanadium, not less than	0.15	" "

As rolled, in plates, shapes, and bars, this steel would have approximately the following properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
60 000	85 000	12%	45%
to	to	to	to
80 000 lb.	110 000 lb.	20%	60%

By the simple operation of heating to 1 100 or 1 150° Fahr., any irregularities in hardness and strength, due to variation in rolling temperatures or uneven cooling, can be removed without appreciably decreasing the elastic limit and tensile strength, and, at the same time, increasing the ductility. This operation would not be an expensive one, and experience might show that it would be unnecessary for this grade of steel.

Type "A" chrome vanadium steel can be sheared, punched, reamed, bent, etc., without any considerable increase of shop manipulations.

Much higher elastic limit and tensile strength can be obtained from this steel by heat treatment, quenching and tempering, but this would increase the shop manipulations very materially, necessitating drilling instead of punching and reaming. It would also add materially to the cost. Any bends would have to be made before heat treatment.

Rather than heat-treat steel of this type, it would be preferable to attain elastic limits of 80 000 to 100 000 lb. by increasing the carbon percentage, and normalize the steel by heating to 1 100 or 1 150° Fahr.

Mr. Norris.

The range in chemical composition for this grade would be:

Carbon	0.30	to	0.40	per	cent.
Manganese	0.40	to	0.60	"	"
Chromium	0.60	to	0.90	"	"
Phosphorus, not more than			0.05	"	"
Sulphur, not more than			0.05	"	"
Vanadium, not less than			0.15	"	"

Plates ½ in. thick, in the upper range of this composition, have shown the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 2 in.	Reduction of area.
125 000 lb.	145 000 lb.	18%	58%

These plates could be punched and sheared as rolled, but it would be advisable no doubt to normalize, as before described, and also to drill rather than to punch the holes.

Plates ⅝ in. thick, at the low range of this composition, have shown the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 2 in.	Reduction of area.
83 000 lb.	116 000 lb.	20%	45%

Normalized, this grade of steel should give, in plates and shapes, the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
75 000 to 100 000 lb.	100 000 to 125 000 lb.	More than 12%	More than 50%

The cost of these chrome vanadium steels would be about 3 cents per lb. more than for ordinary carbon steel.

The writer believes that a simple carbon vanadium steel would prove commercially more attractive than the vanadium steels containing chromium or nickel or both. It is possible to obtain a very material increase in elastic limit by the addition of vanadium to a simple carbon steel, and such steels can be manipulated almost, if not quite, as readily as ordinary carbon steel.

Tests from $1\frac{1}{16}$-in., round, rolled bars of acid open-hearth casting steel of the following composition give a very good idea of what can be expected from this type of steel and how it compares with simple carbon acid open-hearth casting steel:

Carbon	0.26	to	0.27	per	cent.
Manganese	0.60	to	0.64	"	"
Silicon	0.25	to	0.25	"	"
Vanadium	0.21	to	0.00	"	"
Phosphorus, less than	0.05	to	0.05	"	"
Sulphur, less than	0.05	to	0.05	"	"

Mr. Norris.

Elastic limit ....70 000 to 56 000 lb.
Tensile strength .....88 000 to 72 000 "
Elongation in 2 in.......28.5 to 34 per cent.
Reduction of area.......57.5 to 58 " "

From these and other tests, it would seem feasible to specify as follows, for this type of steel:

Carbon ....0.20 to 0.30 per cent.
Manganese ....0.60 to 0.80 " "
Phosphorus, not more than.........0.05 " "
Sulphur, not more than..........0.05 " "
Vanadium, not less than..........0.15 " "

Elastic limit ....50 000 to 70 000 lb.
Tensile strength ....80 000 to 100 000 "
Elongation in 2 in., more than......20 per cent.
Reduction of area, more than......45 " "

The additional cost of vanadium steel of this grade over that of ordinary carbon steel should not be more than 1 cent per lb., and very likely would be less.

For built-up members of vanadium steel, the writer believes that vanadium steel rivets should be used, in order to utilize more fully than would be otherwise possible the high physical qualities of the vanadium steel shapes and plates. The rivets could be either of chrome vanadium steel or simple carbon vanadium steel. The composition for chrome vanadium steel rivets should be:

Carbon ....0.15 to 0.20 per cent.
Manganese ....0.30 to 0.50 " "
Chromium ....0.40 to 0.60 " "
Phosphorus, not more than...0.05 " "
Sulphur, not more than......0.05 " "
Vanadium, not less than......0.15 " "

This steel, as rolled in rounds, would have the following properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
50 000 to 65 000 lb.	70 000 to 90 000 lb.	More than 18%	More than 50%

Single and double shear tests of rivets of steel of this type show:

For single shear......20 per cent.
For double shear......30 " "

greater than for ordinary rivets with a tensile strength of 55 000 lb.

Mr. Norris.

The simple carbon vanadium steel should have a chemical range of:

Carbon ....0.15 to 0.20 per cent.
Manganese ....0.60 to 0.80 " "
Phosphorus, not more than....0.05 " "
Sulphur, not more than.......0.05 " "
Vanadium, not less than......0.15 " "

This steel, as rolled in rounds, would have the following properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
40 000 to 55 000 lb.	65 000 to 85 000 lb.	More than 18%	More than 50%

There should be no difficulty in driving rivets of either of these types of steel.

In the case of eye-bars, possibly the conditions are more favorable for the use of alloy steels than for built-up members. They can be more readily and advantageously heat-treated to develop high elastic limits.

In 1909 a number of tests were made of full-sized eye-bars, heat-treated, of chrome vanadium and chrome nickel vanadium steel. These eye-bars were made from bars 14 by 2 in., had 34-in. heads with 12-in. pin-holes, and were 25½ ft. long over the pin-holes.

The chrome vanadium steel bars experimented with were too high in carbon, and the results obtained for elongation in 20 ft. were not quite as good as in the case of the bars from the chrome nickel vanadium steel. Tests from this latter steel gave results ranging as follows, depending on the drawback or annealing temperature after quenching:

	No. 1.	No. 2.
Elastic limit	63 280 lb.	80 480 lb.
Tensile strength	93 500 "	99 800 "
Elongation, 12 in.	35 per cent.	32.5 per cent.
Elongation, 20 ft.	14.2 " "	7.9 " "
Reduction of area	50.8 " "	52.3 " "

Tests on 2 by ½-in. specimens turned up from the disks cut out in machining the eyes check the elastic limit and the tensile strength obtained from the full-sized bar very well. A test from the eye disk of Bar No. 2 showed:

Elastic limit ....83 040 lb.
Tensile strength ....94 140 "
Elongation, 2 in........... 25 per cent.
Reduction of area.......... 71.9 " "

Mr. Norris.

The chemical composition of this steel was approximately:

Carbon ....0.25 per cent.
Chromium ....0.90 " "
Nickel ....1.20 " "
Vanadium ....0.17 " "

Based on these tests, it is perfectly feasible to specify as follows for heat-treated eye-bars of chrome vanadium or chrome nickel vanadium steel:

Elastic limit ....65 000 to 80 000 lb.
Tensile strength ....85 000 to 105 000 "
Elongation in 2 in......................20 per cent.
Reduction of area........................50 " "

The cost of heat treatment for eye-bars would probably be from ¼ to ⅜ cents per lb. The cost per pound of the chrome vanadium or chrome nickel vanadium steel would be about 3 cents more than that for carbon steel.

The writer believes that eye-bars made from simple carbon vanadium steel of the following composition, either heat-treated, quenched and annealed, or normalized, heating to about 1 100 or 1 150° Fahr., after the heads have been forged, will give elastic limits approaching those of the more expensive chrome or chrome nickel vanadium steels:

Heat-treated or normalized:
Chemical composition.

Carbon ....0.30 to 0.40 per cent.
Manganese ....0.60 to 0.80 " "
Phosphorus, not more than.......0.05 " "
Sulphur, " " " ....0.05 " "
Vanadium, not less than..........0.15 " "

Physical properties.

Elastic limit ....60 000 to 75 000 lb.
Tensile strength ....85 000 to 100 000 "
Elongation in 2 in., more than.....18 per cent.
Reduction of area, more than......45 " "

The writer has given considerable attention to simple carbon vanadium steel containing from 0.60 to 0.80% of manganese. This steel will be the commercial or every-day vanadium steel of the immediate future. It is much cheaper than the vanadium steels containing chrome or nickel. It is at least 40% better than simple carbon steel of otherwise the same composition, and is bound to be used extensively in the near future for rails, general and locomotive forgings, and special structural purposes. It presents no special difficulties in manufacture over simple steel.

Mr. Norris. The quantity of vanadium which remains in the steel is about 80% of that added. It is very evenly distributed. No instances of segregation are known, and it has a strong influence in overcoming the segregation of other elements, particularly carbon.

Vanadium will readily alloy with nickel, and better results can be obtained from a 2% nickel vanadium steel than from a 3½% straight nickel steel of the same carbon content.

There would be absolutely no advantage gained in the use of titanium with vanadium. Vanadium is an alloying metal and is used as such, not as a scavenger or deoxidizer. It is only the vanadium which alloys with the steel that can be considered as influencing the quality of the latter. Titanium has a little merit, as a deoxidizer, over silicon or aluminum, and its action is apparently a surface reaction. It is a question whether the observable reaction, when evident, is not with the highly oxidizable basic slag with which it comes in contact, or even with the atmosphere.

Mr. Richards. J. W. RICHARDS,* ESQ. (by letter).—The statements of Mr. Le Conte concerning the properties of aluminum steel do not agree with the rather extensive investigations of Sir Robert Hadfield, as set forth in a paper† before the Iron and Steel Institute. In that paper and its discussion it was shown that additions of 2¼% of aluminum to 0.22 carbon steel had very little effect on the elastic limit or breaking strength, but reduced greatly the elongation and reduction of area, that is, it makes the metal more brittle.

The paper concludes by recommending the advisability of adding not more than 0.10 or 0.15% of aluminum to steel, with the object, not of making an alloy, but that the addition may be consumed in de-oxidizing the steel. This is, in fact, the proper and valuable function of adding aluminum to steel. Up to 0.1% is added to Bessemer steel, 0.01 to 0.05% to open-hearth steel, and 0.001 to 0.005% to crucible or electric furnace steel. This may be called universal practice in the steel industry, for in practically every large steel works such minute additions of aluminum are now made to ensure complete de-oxidization and produce sounder castings.

Mr. Waddell. J. A. L. WADDELL,‡ M. AM. SOC. C. E. (by letter).—The writer's hearty thanks are herewith extended to all who have done him the honor to discuss this paper, and he is specially gratified by the appreciative reception which it has met at their hands.

In this résumé he will treat only of those topics on which a disagreement of opinion has been expressed, or those which afford data for a further development of the subject. Although many of the points raised in several of the discussions are foreign to the subject at issue, dealing with bridge designing and construction in general,

* Consulting Metallurgist, Aluminum Company of America, New York City.

† *Journal*, Iron and Steel Inst., 1890, Vol. II, p. 161.

‡ Kansas City, Mo.

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nevertheless, the writer has answered such points to the best of his ability and according to his experience.

Replying to the suggestion by Mr. Le Conte, which has been offered also by Mr. Molitor, that experiments be made with aluminum as an alloy material for bridge steel, the writer regrets that hitherto he has been unable to collect any authentic information concerning aluminum-steel alloys; nor did he pay much attention to the possibilities of such a combination on account of the previous high price of aluminum. With a present price of only 20 cents per lb., however, and the possibility of attaining an elastic limit of 100 000 lb. per sq. in., the conditions are changed materially, making the expenditure of much time, effort, and money on the necessary investigation well worth while. Mr. Le Conte stipulates that before adding the cake of aluminum to the molten steel, the latter must first be " 'thoroughly purified' and its elastic limit raised thereby to 60 000 lb. per sq. in." How this is to be done, he does not say. If steel, high in carbon, produced by the electro-metallurgical process, were used, such a great elastic limit might possibly be attained; but, judging from what was told the writer by some gentlemen interested in titanium, the use of that metal as a scavenger does not materially raise the elastic limit or the ultimate strength of the steel, its effect being mainly to make the product more uniform and reliable.

It is evident that much experimenting would have to be done in order to determine how best to produce an aluminum-steel alloy suitable for bridgework.

Mr. Le Conte's estimate of 2.5 cents per lb. for the excess cost of the manufactured alloy appears to be liberal enough; for purified steel smelted by the electro-metallurgical process, as claimed by its French producers, costs only 0.9 cent per lb. more than carbon steel, which amount added to the cost of the aluminum, would make the excess only 1.5 cents per lb., leaving 1 cent per lb. to cover a possible slightly increased cost of rolling the plates and shapes and the extra profit which, doubtless, would be insisted on by the manufacturers on the plea of expense due to innovation from current practice.

All that Mr. Petinot has said in his discussion concerning the use of titanium in the manufacture of steel is of the greatest interest and value to the Engineering Profession; but it is to be regretted that he omitted to state a number of facts of importance which he mentioned to the writer during a long conversation on the subject held a day or two previous to the presentation of the paper to the Society. At the moment of writing, the writer cannot recall the omitted information, except only that (as he indicated in the comment on Mr. Le Conte's discussion) titanium purifies and makes much more reliable the steel with which it is alloyed, but does not increase to any great extent the elastic limit or the ultimate strength.

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The writer is glad to note the corroboration of the claim made by Mr. T. L. Willson to the effect that it is practicable to manufacture satisfactory nickel steel by the addition of ferro-nickel instead of the pure metal. It seems strange that Mr. Willson has not seen fit to take part in the discussion of this topic; for he has been given several opportunities to do so.

As, according to Mr. Petinot, the cost of purifying steel by the use of titanium amounts to only 25 or, at most, 50 cents per ton, it would seem advisable, after his claims have been well established, for the Profession to insist on its use in the manufacture of all carbon steel for bridgework.

The writer desires to emphasize forcibly the remarks of Mr. Moisseiff concerning the necessity of proportioning all highway bridge floors so as to carry properly the heavy concentrated live loads imposed on them by automobile trucks, and concerning the inefficiency of buckle-plates to support pavements subject to such loads. Every bridge designer should take these statements to heart, and be guided thereby in detailing his floors for highway structures; for, no matter how remote or insignificant a highway bridge may be, it is within the realm of possibility that at any time it may have to carry heavy automobile trucks.

Concerning Mr. Moisseiff's warning about the greater distortions (and, therefore, greater secondary stresses) in bridge members when high-alloy steel is used: This is evidently well founded; but, on the other hand, the fact that the members are more slender in an alloy-steel bridge than in a corresponding carbon-steel structure offsets somewhat the injurious effects of the extra distortion. Moreover, in the best modern bridge engineering practice, the greater part of the maximum secondary stress on each main member is overcome by forcing a reverse secondary stress into the metal during erection.

Concerning the efficiency of nickel-steel rivets, the writer would refer Mr. Moisseiff to the discussions by Messrs. Hodge and Skinner at the meeting of the Society when the paper was presented, only a part of which has been recorded in print in Mr. Skinner's discussion. It seems from these discussions that even very long and large nickel-steel rivets can be driven satisfactorily in both shop and field. In alloy-steel bridges, as in carbon-steel bridges, it will, necessarily, be the practice to use much softer steel for rivets than for plates and shapes in order to make possible the cutting out of defective rivets.

Mr. Skinner's question as to the possibility of developing a super-strong alloy steel by subjecting finished members to heavy test loads at the shops, opens up a new subject. The writer thinks that such a practice might be feasible for eye-bars, in which the holes might be drilled small for testing and be enlarged after the bars are stretched; but that kind of manipulation could not well be applied in the case

of compression members, unless they are all pin-connected, which construction, although possible, would involve many complications and difficulties in detailing. Mr. Waddell.

Mr. Molitor's remarks concerning piping and segregation are both pertinent and timely; and it cannot be questioned that the manufacture of "high-alloy" steels for bridges should receive far better attention than that of the commercial carbon steel for such structures. Manufacturers object to any requirement that tends to prevent them from turning out their product quickly and in great quantities. One cannot blame them for this, because it is human nature to desire to make as great a profit on work as possible; but if engineers would insist on obtaining what they know to be necessary in order to secure the best results, and if they would make their clients pay a proper price for the improvement in manufacture, it should not be very difficult—at least not impossible—to effect important innovations in the art of bridge building.

It is this reluctance of the metal manufacturers to adopt innovations and to take special pains, which has prevented the Engineering Profession from having had at its disposal during the last five or six years a nickel steel for bridge building with a minimum elastic limit of 60 000 lb. per sq. in. A little extra care in the furnace manipulation and a moderate increase in the cost of shopwork are all that is necessary to provide such an alloy in as large quantities as may be desired; and the excess pound price for the finished material should only be in direct proportion to the actual excess cost of preparing the rolled sections and of manufacturing them into bridges.

In reference to the theory of segregation and piping, it is to be hoped that engineers will insist on metal manufacturers giving the "Liquid Forged Steel Process" a fair trial. If it be true that the use of this process by purifying the steel and by avoiding piping will raise the elastic limit and ultimate strength of the metal 20%, a most important advance can be made by its adoption in the attainment of a true high-alloy steel for bridge building.

The writer will have to take issue with Sir Bradford Leslie concerning the effect of impact on bridges. Sir Bradford appears not to believe that impact from rapidly moving live loads on bridges is an actuality. The writer knows that it is, for he has both seen and measured it on a number of occasions. In one series of experiments on spans as a whole (not on their individual members), he found increased deflections from motion varying from 20% for spans of 160 ft. to 50% for a span of 88 ft. It is true that the longer spans were pin-connected and of ancient design, and that the shortest one was of the riveted type and disgracefully detailed; nevertheless, other experiments of his on properly designed bridges all show more or less increased deflections due to motion. Again, a committee of

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the American Railway Engineering and Maintenance of Way Association has made many hundreds of determinations of impact on bridges; and its results are a matter of record.

Sir Bradford questions the writer's reason for assuming that long-span railway bridges will have, at least, two tracks. He would, indeed, be a short-sighted American engineer who, to-day, would counsel his client to build a bridge of very long span with a single track; although it would be quite legitimate to adopt the writer's patented expedient of designing it in such a manner that, in the future, duplicate trusses might be added so as to double its carrying capacity.

It is not a dread of wind pressure, as Sir Bradford supposes, which causes American bridge engineers to double-track their long-span bridges (for the width between trusses can be increased to any desired amount without providing an additional track), but because they recognize the fact that if the client can afford to spend a large sum of money for a railroad bridge, the traffic of the line should be great enough to call for a double track, if not at present, at any rate in the near future.

Mr. Fergusson's discussion is really based on the writer's former paper, "Nickel Steel for Bridges",* as it hardly deals at all with the question of using high-alloy steels. Moreover, Mr. Fergusson is apparently somewhat behind the times; for, in several instances, he questions established facts.

He says that "the use of nickel steel cannot be adopted, generally, as the best material for all classes of bridgework." All that prevents this to-day is the unwillingness of the manufacturers of bridges to adopt the innovation and to put a reasonable price on the product.

He says, also, that "carbon steels, for a tensile strength of 40 tons per sq. in., have advantages over nickel steels carrying low percentages of nickel." Any ordinary carbon steel having a minimum ultimate strength of 40 tons per sq. in. would be too brittle to use in bridge construction. It is true that, by taking special pains in its manufacture and drilling solid all the rivet holes, it might be practicable to use such high carbon steel; but it would be bad policy to do so when a nickel steel some 40 or 50% stronger is obtainable at a not entirely unreasonable price. Nickel steel having an ultimate tensile strength of 60 tons is more uniform and reliable than an ordinary carbon steel of 40 tons—at least, that is the writer's opinion, based on an extended experience with bridge metals.

Mr. Fergusson says "the testing of nickel steel during the process of manufacture is very complicated, the material is subject to great variations;" etc.

Such is not the writer's experience in the building of nickel-steel bridges, nor, unless he is greatly mistaken, is it that of Mr. Hodge

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101.

or of other American engineers who have had occasion to build structures with that alloy. Mr. Waddell.

Mr. Fergusson says:

"To use a nickel steel of very high tensile strength, chiefly to save weight in the bridge material—unless where the conditions are imperative—would seem to be extravagant, and even unwise. There must be risk in trusting to the highest limit of tensile strength of a manufactured steel which is known to be very variable."

In answer to this the writer would state that nickel steel is not "very variable", as can be seen by a systematic study of his inspectors' reports, which are published in Appendix A of the paper on "Nickel Steel for Bridges." Those records show clearly that the different kinds of nickel steel are just as regular in their elastic limits and ultimate strengths as the different kinds of carbon steel.

There is no more risk in "trusting to the highest limit of tensile strength" of nickel steel than in trusting to the highest limit of tensile strength of carbon steel—moreover, it is not the highest limit which is adopted as a minimum in specifications, but one fairly close to the lowest limit, so chosen as to avoid the necessity for many rejections.

The writer wonders what Mr. Fergusson deems is the object of bridge engineers in ever using nickel steel for their structures, unless it be "to save weight in the bridge material." That is the sole *raison d'être* for its adoption.

Mr. Fergusson says: "The proportion of 3.5% of nickel, which he [the writer] gives, appears to be too low." For plate-and-shape steel this is as much as it is either profitable or advisable to use. Any greater percentage would add to the expense of the rolled metal and increase the cost of the shopwork. However, for eye-bars in which the amount of shopwork is small, it is economical to increase the percentage of nickel to about 4.25.

Mr. Fergusson says: "As he [the writer] does not mention carbon in his analysis, it may be presumed that he has eliminated it; surely this is a difficult operation in commercial manufacture?"

It certainly would be—but, unfortunately for Mr. Fergusson's deduction, the writer did not attempt to cut out the carbon—far from it. The great strength of nickel steel in bridgework is largely due to the fact that the addition of nickel permits of a substantial increase in the percentage of carbon. The nickel itself adds to the strength of the metal, but at the expenditure of considerable money; while the increase in the carbon content adds to the said strength without augmenting the cost at all. Surely Mr. Fergusson has failed to read with any care the paper on "Nickel Steel for Bridges"; for if he had done so, he would not have made such a blunder. The specifications

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for the various classes of nickel steel given in Table 4 of that paper call for the following compositions:

Ingredients.	Percentages.		
	Rivet steel.	Plate-and-shape steel.	Eye-bar steel.
Nickel................	3.50 (3.25 to 3.75).	3.50 (3.25 to 3.75).	4.25 (4.00 to 4.50).
Carbon.........	0.15 (0.12 to 0.18).	0.38 (0.34 to 0.42).	0.45 (0.40 to 0.50).
Phosphorus..........	0.03 maximum.	0.03 maximum.	0.03 maximum.
Sulphur...............	0.04 maximum.	0.04 maximum.	0.04 maximum.
Silicon	0.04 maximum.	0.04 maximum.	0.04 maximum.
Manganese...........	0.6 (0.55 to 0.65).	0.70 (0.65 to 0.75).	0.80 (0.75 to 0.85).

Speaking of the strengths of nickel steels of varying percentages of nickel, Mr. Fergusson states that "the increase of nickel requires a decrease of carbon, together with an increase of manganese, otherwise, although the breaking load may be the same, the value of the elastic limit will fall rapidly." Within the limit of the writer's experiments, which do not cover alloys higher in nickel that 4.5%, he has found that just the opposite condition exists; for increasing the carbon content raises both the elastic limit and the ultimate strength.

The writer does not claim that nickel steel having a greater percentage of nickel than 3.5 is absolutely unworkable in the shops; but that commercially it is so. On page 103 of the paper on "Nickel Steel for Bridges" will be found the following:

"Curiously enough, the superior limit of nickel steel for workability in bridge shops varies but little, if any, from the economic amount based on present prices of steel and nickel. For plates and shapes, which form the principal part of bridge superstructures, any materially greater percentage than 3½ renders the alloy too refractory for the various shop manipulations to which it would be subjected in manufacturing it into bridges, although superstructures built of steel still higher in nickel would be perfectly safe and satisfactory for operation."

If Mr. Fergusson will read the results of the investigations of Messrs. Carpenter, Hadfield, and Longmuir,* he will see the danger of using high percentages of nickel in bridge steel; for these gentlemen have demonstrated the existence of a "brittle zone" in nickel steels, beginning with a percentage of nickel lying between 4.25 and 5.0, and ending at 20. These investigations were referred to in the writer's former paper.

Mr. Fergusson points out what he claims to be "two disadvantages in nickel steel for bridgework:

"(*a*) The difficulty of welding, and working;

"(*b*) The necessity of keeping the carbon extremely low—under 0.20%—whereas the fatigue resistance of steel is increased enormously by raising the percentage of carbon."

* *The Engineer* (London). November 24th, 1905.

To these criticisms the writer would reply as follows: **Mr. Waddell.**

First.—Welding in bridgework is now a thing of the past, and has been ever since the days of wrought-iron bridges.

Second.—There is certainly more difficulty in manipulating nickel steel in the shops than in handling carbon steel there, but the extra cost is trifling, amounting to between 0.15 and 0.25 cent per lb. This is a bagatelle compared with the advantages gained by the use of the alloy.

Third.—There is no reason whatsoever for "keeping the carbon extremely low."

Fourth.—The fatigue of steel in bridges is now conceded by the leading American engineers to be entirely a myth. For many years it was a great bugbear to bridge builders; but to-day it is a well-known fact that there is no "fatigue" of metal unless the elastic limit is exceeded; and most of the reputable American bridge engineers aim to keep the actual intensities of working stresses in their structures down to about one-half of the elastic limit.

Answering Mr. de Broë's discussion, in reference to the price of steel: the writer assumed in 1907 that the average pound price for plate-girder spans erected was 4.0 cents; and to-day it is half a cent less, or 3.5 cents. Subtracting from this, as an average, 1.25 cents for freight and erection, would make the pound price, f. o. b. cars at the shops, 2.25 cents, which is about right. During bad times in the United States plate-girder spans can be bought for less than 2.0 cents per lb., f. o. b. cars at the works; but when times are very good, this figure is increased some 50% or more. Mr. de Broë's quotation of £13/10/0 per ton, as the highest price paid in England during the last few years for bridge superstructures, corresponds to nearly 3.0 cents per lb. The writer does not think that bridges can be manufactured in England as cheaply as they can in the United States, but admits that in this opinion he may be mistaken.

In regard to the question of impact brought up by Mr. de Broë, the writer has treated that subject already in his reply to Sir Bradford Leslie's discussion; but he would remark that although the recorded impact on that 40-ft. span at 45 miles per hour may truly have been zero, a lower velocity of train might readily have indicated a high percentage—such, at least, has been the experience of American engineers (among others the writer) in testing bridges for deflection under static and moving loads. The writer is quite convinced from a practical experience in the designing, building, examination, and testing of bridges, extending over a period of more than 33 years, that the use of proper impact formulas or curves is a necessity in truly scientific design.

He takes pleasure in complying with Mr. de Broë's request for "the more accurate formula" which he has evolved from the published

Mr. Waddell. tests of a commission of the American Railway Engineering and Maintenance of Way Association appointed to experiment on impact from moving loads on railroad bridges. The formula for coefficients is

$$I = \frac{165}{L + 150},$$

or, in terms of percentages, it is

$$I = \frac{16\,500}{L + 150},$$

where L is the length of span, in feet, covered by the moving load. The highest of the three curves in Fig. 28, prepared from this formula, gives the coefficient of impact for all spans between zero and 1 000 ft. At these limits, the coefficients are, respectively, 1.10 and 0.14.

The writer regrets that he cannot at the present writing give Mr. de Broë the name of the publication containing the results of the impact experiments from which the foregoing formula was prepared.

The discussion by Mr. Norris is exceedingly valuable, being directly to the point of the main question at issue. It comes as a reply to a long letter by the writer addressed to the American Vanadium Company, inviting its engineers to discuss his paper, and containing the following questions and suggested topics:

"A.—Suitability of vanadium steel for eye-bars. Give probable elastic limit, ultimate strength, elongation, reduction of area, composition, etc., of the finished bars, also their approximate cost per pound.

"B.—Suitability of vanadium steel for built members of bridges, giving the same information as in A.

"C.—Can the treated vanadium steel be manufactured into built members of bridges without losing the great effect of the treatment? Can the treated vanadium steel be sub-punched and reamed, or would it have to be drilled solid? Can it be bent cold without injury, or must it be heated, bent, and re-tempered? If the latter, will it keep its shape during the re-tempering so that the pieces will fit when assembled?

"D.—What kind of rivets would you use in connecting the components of built members made of vanadium steel? If you use vanadium steel rivets, what would be their composition?

"E.—Would you advise the use of chromium for bridge steel, and, if so, how much?

"F.—How much vanadium do you put in the charge, and how much remains in the metal? How evenly is the vanadium distributed throughout the metal?

"G.—What can you tell me about alloying nickel and vanadium for bridge steel? What about alloying nickel, chromium, and vanadium?

"H.—Do you think the addition of titanium would do any good? I understand that it acts only as a purifier, and passes off in the slag.

"I.—Are there any other substances than those that I have mentioned which you think might profitably be alloyed with vanadium for bridge steel, and how would you use them?

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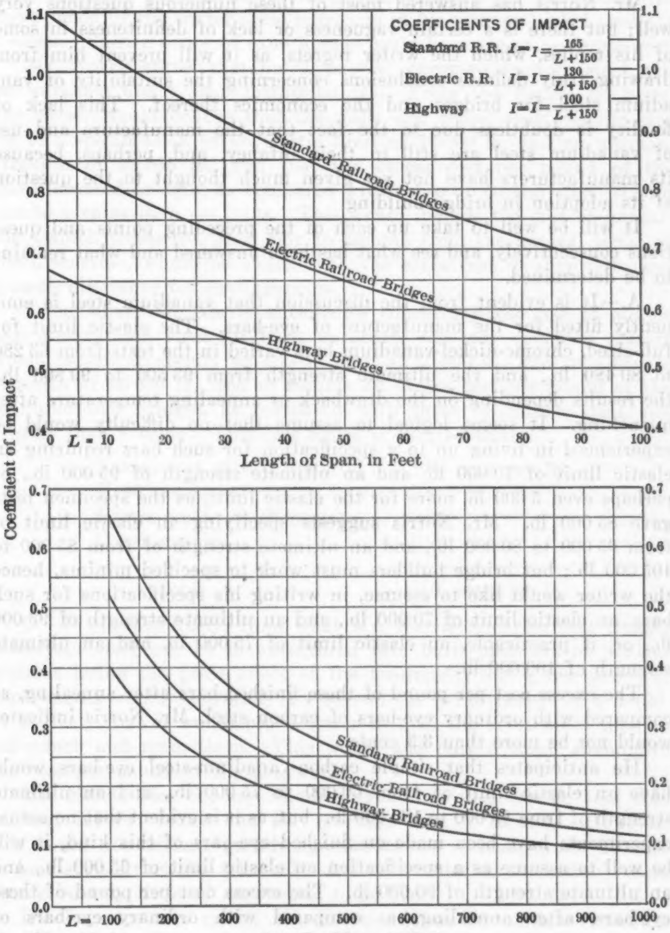

FIG. 28.

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"J.—What specifications would you suggest for vanadium steel, both untreated and treated—in reference to bridgework, of course?"

Mr. Norris has answered most of these numerous questions very well; but there is a certain vagueness or lack of definiteness in some of his replies, which the writer regrets, as it will prevent him from drawing truly definite conclusions concerning the suitability of vanadium steel for bridges and the economics thereof. This lack of finality is doubtless due to the fact that the manufacture and use of vanadium steel are still in their infancy, and, perhaps, because its manufacturers have not yet given much thought to the question of its adoption in bridge building.

It will be well to take up each of the preceding points and questions consecutively, and see what has been answered and what remains to be determined.

A.—It is evident from the discussion that vanadium steel is eminently fitted for the manufacture of eye-bars. The elastic limit for full-sized, chrome-nickel-vanadium bars varied in the tests from 63 280 to 80 480 lb., and the ultimate strength from 93 500 to 99 800 lb., the results depending on the drawback or annealing temperature after quenching. It seems logical to assume that no difficulty would be experienced in living up to a specification for such bars requiring an elastic limit of 70 000 lb. and an ultimate strength of 95 000 lb., or perhaps even 5 000 lb. more for the elastic limit, as the specimen tests gave 83 000 lb. Mr. Norris suggests specifying an elastic limit of from 65 000 to 80 000 lb., and an ultimate strength of from 85 000 to 105 000 lb.; but bridge builders must work to specified minima, hence the writer would like to assume, in writing his specifications for such bars, an elastic limit of 70 000 lb., and an ultimate strength of 95 000 lb., or, if practicable, an elastic limit of 75 000 lb. and an ultimate strength of 100 000 lb.

The excess cost per pound of these finished bars after annealing, as compared with ordinary eye-bars of carbon steel, Mr. Norris indicates would not be more than 3.5 cents.

He anticipates that simple carbon-vanadium-steel eye-bars would have an elastic limit of from 60 000 to 75 000 lb., and an ultimate strength of from 85 000 to 100 000 lb.; but, as it is evident that no actual experiments have been made on finished eye-bars of this kind, it will be well to assume as a specification an elastic limit of 65 000 lb., and an ultimate strength of 90 000 lb. The excess cost per pound of these eye-bars, after annealing, as compared with ordinary eye-bars of carbon steel, the writer deduces from two statements of Mr. Norris to be not more than 1.5 cents.

B.—It is not quite so evident from the discussion that vanadium steel is as suitable for built members of bridges as it is for eye-bars;

nevertheless, it seems probable that it would be found to be satisfactory. To determine this would require a series of experiments, consisting mainly of the various shop manipulations of the metal, like those made by the writer some years ago in his tests on nickel steel for bridges. Such a series of experiments could be made at moderate cost. Mr. Waddell.

Mr. Norris states that his Type "A" chrome-vanadium steel will be workable under shop manipulations. From the figures he gives, the writer would deduce that the mills and shops could readily live up to a specification of an elastic limit of 70 000 lb. and an ultimate strength of 95 000 lb., and that the excess cost per pound of the manufactured bridges, as compared with those of carbon steel, should not be greater than 4.0 cents. This would include the cost of annealing (which treatment, it is fair to assume from the general tenor of the discussion, would be obligatory), the extra cost of shopwork, and a small allowance for contingencies.

From what Mr. Norris anticipates concerning the use of simple carbon-vanadium steel for built members, one may conclude that it would be safe to specify an elastic limit of 60 000 lb., and an ultimate strength of 90 000 lb., and to assume an excess cost per pound of the manufactured bridges, as compared with those of carbon steel, not greater than 2 cents. This last figure is derived thus:

Excess cost of raw material	1.0	cent per lb.
Heat treatment	0.5	" " "
Excess cost of shopwork	0.25	" " "
Contingencies	0.25	" " "
Total	2.0	cents per lb.

C.—It appears likely, from what Mr. Norris remarks, that treated vanadium steel can be manufactured into built members of bridges without losing the great effect of the treatment; but a few practical experiments would determine this beyond the peradventure of a doubt. It is likely that it would prove better to drill the rivet holes than to sub-punch and ream them. This would add only a small amount to the cost per pound of the shopwork.

Mr. Norris does not say whether vanadium steel can be bent cold without injury; hence one may surmise that it cannot. However, this would not militate greatly against its use in bridgework, because in long-span structures (the only kind now under consideration) there should be very little, if any, metal to be bent.

D.—In respect to rivets, Mr. Norris advises for chrome-vanadium steel an elastic limit of from 50 000 to 65 000 lb., and an ultimate strength of from 70 000 to 90 000 lb.; and for simple carbon-vanadium steel an elastic limit of from 40 000 to 55 000 lb., and an ultimate strength of from 65 000 to 85 000 lb. He says "There should be no difficulty

Mr. Waddell. in driving rivets of either of these types of steel." True enough; but there might be great difficulty in cutting them out—and, alas, owing to the frailty of human nature, it is impossible to build bridges without having to remove at least a few rivets—and generally a good many of them.

In his experiments on nickel steel for rivets the writer encountered this difficulty, and he was obliged, in consequence, to lower the carbon content (and hence also the elastic limit and the ultimate strength) in his specifications for rivet nickel steel. Nothing but some thorough experiments on vanadium-steel rivets of varying compositions will determine how high a steel it will be safe to use for such rivets.

Concerning rivets for bridges built of high-alloy steels, it is a foregone conclusion that the ratio of the strength of the alloy-steel rivets to that of carbon-steel rivets cannot be as great as the corresponding ratio of strength of plate-and-shape alloy steel to that of plate-and-shape carbon steel. On this account, in high-alloy steel bridges it will be necessary to use proportionately either more rivets or greater rivet diameters—or both.

E.—Mr. Norris has spoken in favor of the use of chromium for vanadium bridge steel, although he indicates a preference for simple vanadium steel. It is possible that the latter is preferable for moderately long spans and the former for very long ones. The economic study based on the preceding deductions from the discussion, which is to follow herein, will determine this point. The quantity of chromium recommended by Mr. Norris varies from 0.6 to 0.9%, in combination with manganese varying from 0.4 to 0.6%, or 0.8 per cent. The writer would be inclined to go a few points, say, 20, higher with the manganese, as his experiments on nickel steel indicated the advantage of such a policy; but, perhaps, the addition of the chromium might necessitate some reduction in the manganese.

F.—For a number of years it was thought by the Engineering Profession in general that vanadium in steel acted only as a scavenger, none of it remaining in the finished product, but all passing off with the slag. However, it seems that such is not the case, but that about 80% of the vanadium which is added to the charge remains in the finished product.

It seems, according to Mr. Norris, that the vanadium is very evenly distributed through the ingot, and that not only is there no danger whatsoever of its segregation, but also that its presence in the molten metal tends to prevent the segregation of other substances—notably carbon. This is most reassuring, and is a strong point in favor of the use of vanadium steel for bridge building.

G.—Mr. Norris does not say much about alloying nickel with vanadium or about combining nickel, chromium, and vanadium. Such com-

Mr. Waddell.

binations are purely a question of economics; and only elaborate investigations can determine what should be the constituents and their proportions to produce the best and most economic alloy. From what Mr. Norris states, one might anticipate that the gain by adding nickel to vanadium-carbon steel or to vanadium-chromium steel would be either very small or non-existent.

H.—Mr. Norris is quite convinced that the addition of titanium to vanadium steel would be useless. Perhaps the explanation for this is that the 20% of vanadium which passes off into the slag acts as a scavenger, thus obviating the necessity for any further purification—which is the sole function of the titanium.

I.—Mr. Norris does not reply to the question as to whether there are any other substances than those herein mentioned, which might profitably be added to vanadium steel. His reason, probably, is that he does not know, never having tried the addition of such elements. It is only extensive experiments that will furnish an answer to this question.

J.—The specifications for the composition and properties of vanadium-carbon steel and vanadium-chromium steel, suggested by Mr. Norris, are very wide in their range, but they are probably as definite as the present state of the art of manufacturing those alloys will allow.

There is one statement near the beginning of Mr. Norris' discussion to which the author must take exception, namely, "In the case of nickel steel, in order to obtain an elastic limit of 60 000 lb. or more, it is necessary to increase the percentage of carbon to undesirable limits."

The plate-and-shape steel experimented on by the writer in his nickel-steel investigation had an elastic limit slightly in excess of 60 000 lb., and was a perfectly satisfactory metal in every essential particular for bridgework. It is true that its manufacture was harder on the tools than ordinary carbon steel, but that feature simply added slightly to the cost of the shopwork. The metal withstood all the shop manipulations as well as ordinary carbon steel; it was just as reliable in every particular; and there was no danger whatsoever from brittleness.

The economics of Mr. Norris' vanadium-carbon and vanadium-chromium steels, as compared with ordinary carbon steels and with the nickel steels which can be obtained to-day for bridgework at the prices last quoted to the writer by the manufacturers, will now be considered. Using the averages just determined in this résumé, we have for the vanadium-carbon steel an elastic limit of 60 000 lb. and an excess pound price of 2.0 cents as compared with carbon steel; and for the vanadium-chromium steel an elastic limit of 70 000 lb. and an excess pound price of 4.0 cents.

Mr. Waddell. On Figs. 29 and 30 are plotted, for simple-span bridges and for cantilevers, respectively, the weights per linear foot of metal for various spans of carbon steel, vanadium-carbon steel, vanadium-chromium steel, nickel steel for $E = 50\,000$ lb., and nickel steel for $E = 55\,000$ lb.; and on Figs. 31 and 32 are shown, for simple-span bridges and for cantilevers, respectively, the economies of the said five classes of steel considered, as follows:

A.—Carbon steel, $E = 35\,000$ lb.

B.—Nickel steel, $E = 50\,000$ lb., at an excess cost, delivered, of 2 cents per lb.

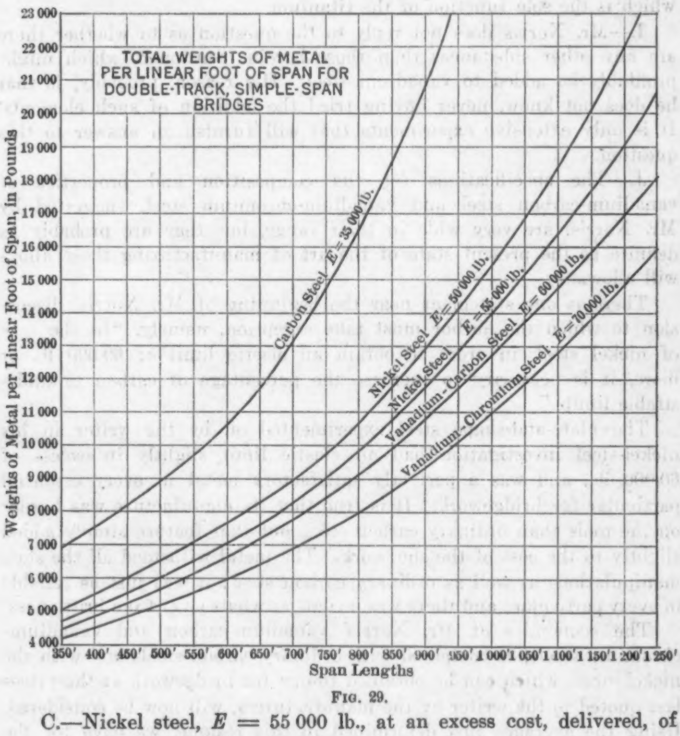


FIG. 29.

C.—Nickel steel, $E = 55\,000$ lb., at an excess cost, delivered, of 2.5 cents per lb.

D.—Vanadium-carbon steel, $E = 60\,000$ lb., at an excess cost, delivered, of 2.0 cents per lb.

E.—Vanadium-chromium steel, $E = 70\,000$ lb., at an excess cost, delivered, of 4.0 cents per lb.

Mr. Waddell.

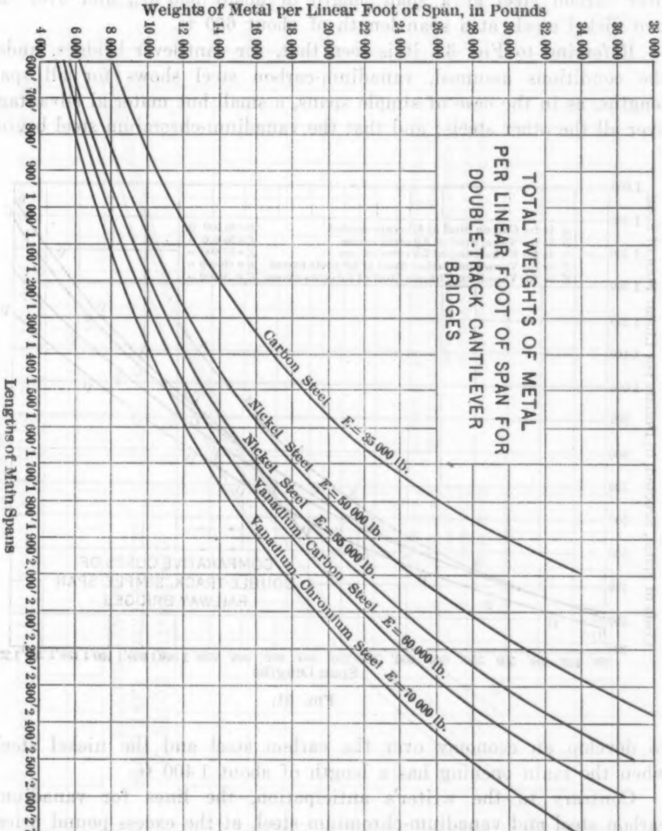


FIG. 30.

Mr. Waddell.

Referring to Fig. 31, it is seen that, for simple-span bridges, under the conditions assumed, vanadium-carbon steel shows, for all span lengths, a small but material advantage over all the other steels; and that the vanadium-chromium steel begins to develop an economy over carbon steel at a span length of about 500 ft., and over the two nickel steels at a span length of about 650 ft.

Referring to Fig. 32, it is seen that, for cantilever bridges, under the conditions assumed, vanadium-carbon steel shows for all span lengths, as in the case of simple spans, a small but material advantage over all the other steels; and that the vanadium-chromium steel begins

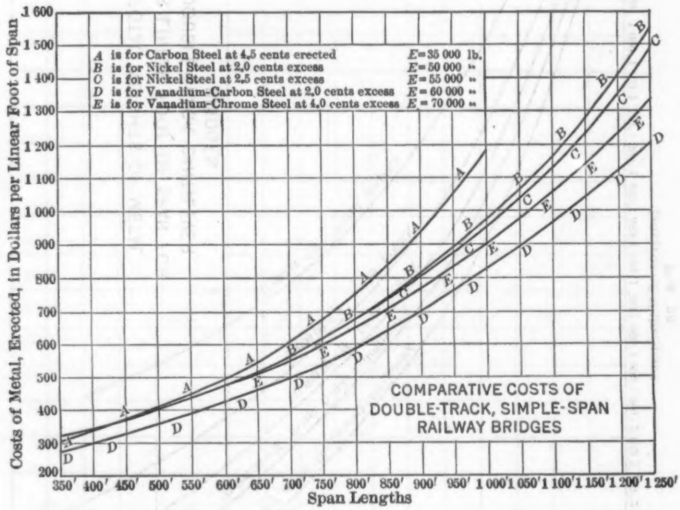


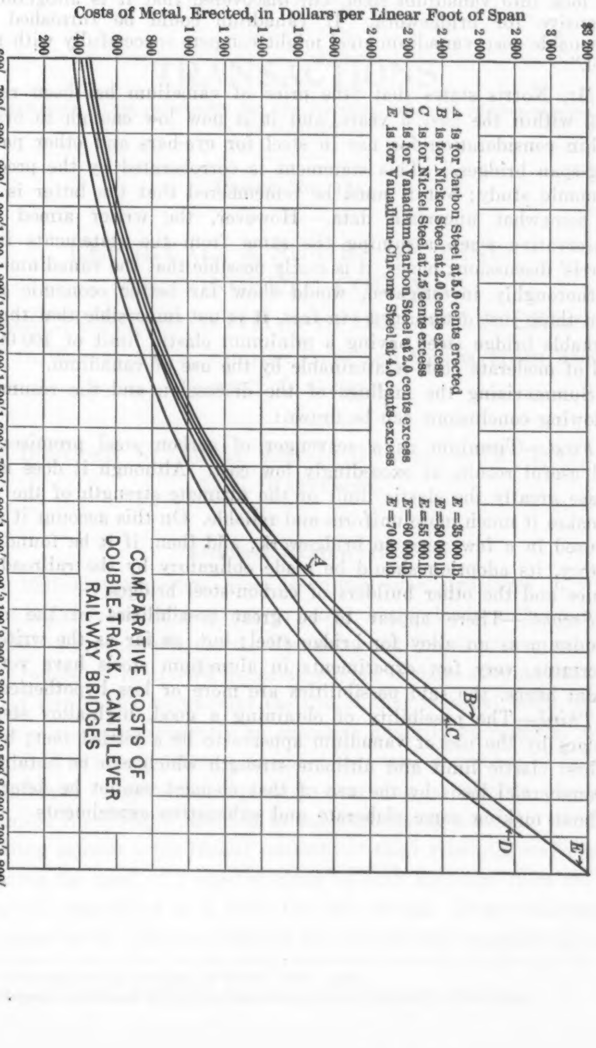
FIG. 31.

to develop an economy over the carbon steel and the nickel steels when the main opening has a length of about 1 400 ft.

Contrary to the writer's anticipation, the lines for vanadium-carbon steel and vanadium-chromium steel, at the excess pound prices assumed, do not cross each other. They would intersect, for cantilever bridges, at a main opening of 2 800 ft., which is about the practicable limit for $E = 70\,000$ lb.

Some 8 or 9 years ago certain leading steel manufacturers quoted to the writer an approximate price of 8 cents or more per lb. for vanadium steel bridge superstructures, delivered f. o. b. cars at the

Mr. Waddell.



Lengths of Main Spans

FIG. 32.

Mr. Waddell. shops. It was this which caused him to make the following statement in his paper on "Nickel Steel for Bridges":

"If there is any other alloy of steel that will give as good results as nickel steel at less cost, the Profession ought to know it. The writer did look into vanadium steel, but discovered that it is altogether too expensive for bridgework. If vanadium could be furnished at a reasonable cost, vanadium steel might compete successfully with nickel steel."

Mr. Norris states that "the price of vanadium has been reduced 60% within the past 3 years, and it is now low enough to bring it within consideration for use in steel for eye-bars and other parts of long-span bridges." This statement is corroborated by the preceding economic study; but it must be remembered that the latter is based on somewhat uncertain data. However, the writer aimed to be conservative when assuming the same from the statements in Mr. Norris' discussion; hence, it is easily possible that the vanadium steels, if thoroughly investigated, would show far better economic results than those just determined—in fact, it is not impossible that the ideal workable bridge steel having a minimum elastic limit of 100 000 lb., and of moderate cost, is attainable by the use of vanadium.

Summarizing the findings of the discussion and the résumé, the following conclusions can be drawn:

First.—Titanium as a scavenger of carbon steel promises good and useful results at exceedingly low cost. Although it does not increase greatly the elastic limit or the ultimate strength of the metal, it makes it much more uniform and reliable. On this account it should be used in a few cases on bridgework; and then, if it be found satisfactory, its adoption should be made obligatory by the railroad companies and the other builders of carbon-steel bridges.

Second.—There appear to be great possibilities in the use of aluminum as an alloy for bridge steel; but, as far as the writer can determine, very few experiments in aluminum steels have yet been made; hence, the said possibilities are more or less hypothetical.

Third.—The possibility of obtaining a good, high-alloy steel for bridges by the use of vanadium appears to be a settled fact; but the highest elastic limit and ultimate strength which can be obtained on a commercial basis by the use of that element cannot be determined without making some elaborate and exhaustive experiments.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 1314

REPORT ON A SERIES OF TESTS ON CONCRETE COLUMNS REINFORCED WITH A SPIRAL OF STEEL.*

By C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie † and C. O. Carey.†

With Discussion by Messrs. Edward Godfrey, A. W. Buel, and C. G. Wrentmore.

General Statement.

In 1907-08 the writers were designing some structures in which certain columns were required to carry heavy loads and would be subjected to considerable shock. The nature of the structures was such that columns of reinforced concrete offered certain advantages, sufficient in importance to warrant the use of that material, even though low stresses and consequently an excessive quantity of material might be required to insure safety. At about this time a number of failures of reinforced columns in important structures at various places in the United States were reported in the engineering journals, sounding a warning against a too liberal estimate of their strength, and plainly indicating the need of a careful study of such members from the experimental standpoint as a basis for safe design. Some experiments then made on the ultimate strength of columns with longitudinal steel

* Presented at the meeting of March 18th, 1914.

† Messrs. Brodie and Carey are Instructors in the University of Michigan.

bars embedded in the concrete indicated that such reinforcement, unless hooped or banded at frequent intervals, could not be relied on to increase the strength of the column, and in some cases might even render its strength less. These experiments, however, were few in number, and the time available did not permit of their being carried out with sufficient care to render the results thoroughly trustworthy; but they served to turn attention to the hooped column, which had been discussed by Considère in his treatise, regarding which experimental information was very meager.

A series of tests on small columns was planned and partly carried out during the first half of 1908, but the departure of Mr. Wrentmore for the Philippines, in July of that year, interrupted the work, and since that time it has been continued by Messrs. Brodie and Carey at irregular intervals, at such times as their University duties would permit.

In all, 115 columns have been made, and usable results have been obtained from 104 of these; the remaining 11 were broken in storage or in handling, were found faulty in construction, or showed such erratic results under tests as to warrant their being thrown out of the list. Those which will be discussed here are Nos. 10 to 22, and 36 to 115, both inclusive, except Nos. 40, 55, 83, 86, 88, 91, 103, 108, 109, and 112, a total of 83 columns. The other 21, making up the 104, include the columns reinforced with longitudinal rods previously referred to, and some special cases which are not germane to the present discussion. The series is still incomplete, some of the sub-groups showing no tests, or only one or two, and others showing wide variations and requiring more tests to determine a dependable average. Moreover, the extensometer tests on a large percentage of the columns were only carried to 700 or 800 lb., and, later, when a comparison of results was made, it was found that it would have been desirable to have carried all tests to at least 1 400 lb. per sq. in. Some columns gave usable results on only one part of the test, as on ultimate strength alone, or extensometer measurement alone; however, in view of the care which has been exercised in carrying out the work, it is believed that the results obtained are worth placing on record, and they are here presented in the hope that they may add in some measure to the sum total of useful information.

Acknowledgment.

The writers take pleasure in acknowledging the assistance received from the Dean of the Department of Engineering of the University of Michigan in granting the use of the University laboratories and equipment for carrying on these tests, to Gardner S. Williams, M. Am. Soc. C. E., for suggestions which were of assistance in securing accuracy and uniformity in the results, and to H. H. Atwell, Assoc. Am. Soc. C. E., and Messrs. J. Schmutz and L. H. Neilson, of the Faculty of Surveying, and Messrs. H. A. Shuptrine, E. Olmstead, L. R. Manville, and M. D. Bensley, students in Engineering, all of the University of Michigan, who gave their time and work to assist in carrying on the tests.

Aim of the Tests.

The tests were planned to determine the longitudinal and lateral deformations of the columns under various loads, increasing from zero to something above the probable maximum working stress, with varying quantities of reinforcement and at different ages, also the ultimate strength and the load at the first visible sign of failure. From the deformation could be determined the change in unit length for various loads, the modulus of elasticity, and Poisson's ratio. Comparison of the results should show the influence of the reinforcement on these several quantities.

Description of the Columns.

All columns were approximately 4 in. in diameter and 27 in. long, over all, cast in a galvanized-iron mould set on a wooden bottom. The mixture throughout was the same, 1 : 2 : 4 by loose volume. The cement was Peninsular brand, bought from a dealer from stock, in fair condition, and as ordinarily used in construction. It was sampled and tested as for construction work, passing satisfactorily the standard tests recommended by the Special Committee (of the American Society of Civil Engineers) on Uniform Tests of Cement. The sand was bank sand from a drift deposit at Ann Arbor, secured from a contractor who made a business of furnishing it to the building trades, and was of a grade acceptable for first-class work. The granulometric composition is shown by Table 2. The coarse aggregate was broken limestone from the quarries at Monroe, Mich., and was passed through a $\frac{3}{4}$-in. and retained on a $\frac{1}{4}$-in. screen.

The mixing was done by hand, and under the personal supervision of the writers. In order to fill well about the spirals, it was necessary to use a wet mixture; it was too wet to be tamped with a small rod, but not so wet as to cause the materials to separate in pouring.

The reinforcement in Columns 10 to 14 was furnished by a commercial firm, built up and ready to be placed in the forms. The spiral was of high-carbon steel, $\frac{1}{8}$ in. in diameter, spaced as shown in Table 3. The spacing members were four $\frac{1}{8}$ by $\frac{1}{16}$-in. steel bars. No tests were made to determine the physical constants of this reinforcing material. These columns were tested only for ultimate strength and first sign of failure, and, for purposes of comparison, are included with similar columns tested at other laboratories.

The reinforcement in Columns 15 to 22 was wound on the concrete shaft, after casting, by placing the column in a lathe and winding the wire on under constant tension—sufficient to bed it slightly in the concrete. This was done just before testing, at the age of 28 days. The end of the wire was anchored securely to the column very near one end, the tension was applied, and a dozen turns were wound on as close as they would lie in order to insure against failure at the end; the spiral was then spaced by using the lead screw until within 1 in. of the other end; then another dozen turns were run on close together. The wire was then brought under the anchor and fastened, the tension was released, and the wire cut free. On these columns the wire had a diameter of 0.079 in., an area of 0.0049 sq. in., and the tension, as given by a spring balance, was 62½ lb. on some columns and 125 lb. on others, corresponding to unit stresses of 12 750 and 25 500 lb. per sq. in.

In the remaining columns in which reinforcement was used the spirals were made up and set in place in the form, and the concrete was then poured. The longitudinal members of the reinforcement were barely heavy enough to hold the spiral in place; they were strips of No. 24 gauge galvanized iron, $\frac{7}{16}$ in. wide, with notches, $\frac{7}{32}$ in. deep, accurately spaced by machine; the consecutive turns of the spiral were dropped into these notches and fastened by pinching down the lugs formed in cutting the notches. Four such spacing strips were used for each coil, and, as they were so light, considerable care was necessary to insure the proper location of the reinforcement. On final test to destruction, however, all spirals were found to have been spaced and

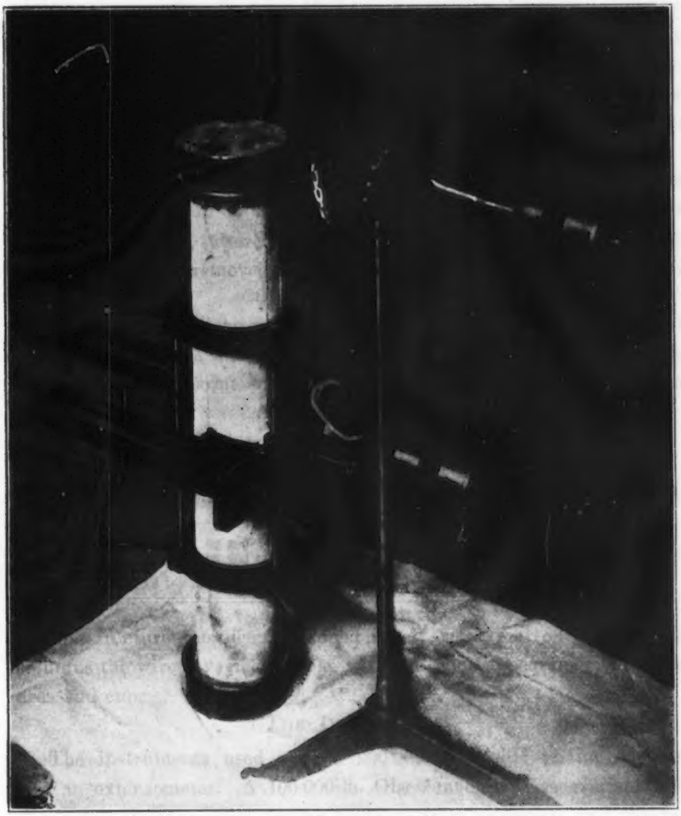

FIG. 1.—EXTENSOMETER.

centered accurately, although in some cases the longitudinal members had taken a spiral position, as may be seen on Fig. 3. A number of extra turns were given at each end of the coil, to make sure that failure should occur in the middle of the shaft, and a circular piece of ¼-in. mesh hardware cloth was placed in each end of the column to give it additional strength. In spite of these precautions, however, a number failed by splitting or crushing at the end, thus failing to develop the full strength of the shaft. The longitudinal spacing strips were not taken into account in estimating the percentage of reinforcement, as they were considered to be too slender to affect sensibly the column under test.

After being cast the columns stood for 24 hours in the forms, and then, except Nos. 10 to 22, were placed in water in the large naval experimental tank, where they remained until removed for testing. Nos. 10 to 22 were removed from the forms at 24 hours and cured in air in the laboratory.

The wire used in the spirals was of unannealed, copper-washed, soft steel, bought from a hardware dealer from stock. Under test it showed an average yield point, ultimate strength, and reduction of area as given in Table 1.

TABLE 1.

Diameter, in inches.	Ultimate strength.	Yield point.	Reduction of area. Percentage.	No. of tests.
0.105	92 330	85 780	50	35
0.079	95 260	87 870	50	45
0.062	78 000	70 000	80	45

The fracture was deeply cupped in all cases, and on crushing the columns the wire invariably broke with the same appearance of reduced area and cup.

THE INSTRUMENTS.

The instruments used were a 200 000-lb. Riehlé testing machine and an extensometer. A 100 000-lb. Olsen machine was available, but the larger clearance, the greater freedom from vibration, and the fact that it was not so frequently in use made the larger machine more desirable. The column was placed in the machine on a cushion of sand in a cast-iron cup which had a machined surface for bearing on the bed of the testing machine.

The extensometer, Fig. 1, was designed by the writers and built in the Instrument maker's shop of the University. The small variation to be measured rendered necessary extreme care and accuracy in construction, and especially the elimination of all lost motion. Knife-edges were ground straight in a Universal grinding machine after tempering, and the V-notches for the knife-edge bearings were made in two parts and similarly ground to true lines. The five rollers of tempered steel were ground to true cylinders, and as nearly of the same size as practicable, and afterward Nos. 2, 3, 4, and 5 were calibrated against No. 1 in a specially designed device.

The small amount of lost motion is shown by the fact that when the column was loaded and the scale-beam rose and fell between its stops a distance of about ¾ in. at the free end, the lateral mirrors traveled to and fro on the scale, showing clearly the increase and decrease of diameter of the column due to the small variation of load produced by this motion of the scale-beam.

The construction of the instrument is shown by Figs. 1 and 2. For longitudinal measurement, there are two cast-iron rings, *q*, connected by three telescoping rods, each made up of the rods, *r* and *s*, and the tube, *t*. The rings, *q*, are fastened to the column by setting up the screws, *w*, to a firm bearing. The rods, *r* and *s*, are then set in their seats in the rings and the nuts lightly screwed down on the spherical heads, as shown in the small sectional view. The tube, *t*, is slipped on and firmly clamped to the short rod, *r*, the longer rod, *s*, being left free to slide within *t* as the column shortens under load. The bar, *u*, attached to *t*, holds the roller between its face and the bar, *s*, being held to a steady pressure by the adjustable spring, *v*. The average of the readings of the three rods gives the deformation of the column.

For lateral measurement, the frame, *a*, with its arms attached, is hung by a chain from the counterbalance arm shown in Fig. 1, by the hook at *g*, which is fast to the sliding bar, *e*, which is moved by *h*. The bar, *e*, and the counterweight on *d* are adjusted until the bar, *a*, hangs level, and the weight on the counterbalance arm is adjusted until it exactly balances the weight of the instrument. It is then adjusted to the column in place in the testing machine, with the knife-edges, *o*, and the screws, *b*, bearing against the small blocks of plate glass cemented to the column. The pieces, *j* and *k*, are bolted rigidly to *a* when in use. The arm, *m*, is held to a firm bearing on its

knife-edges, *n* and *o*, by a spring and plunger in *p*. The distance between these knife-edges is 1 in. and the length from *n* to the end of *m*, on which the roller bears, is 10 in. The roller is placed between the end of *m* and the bar, *l*, the latter being held by a spring to a firm

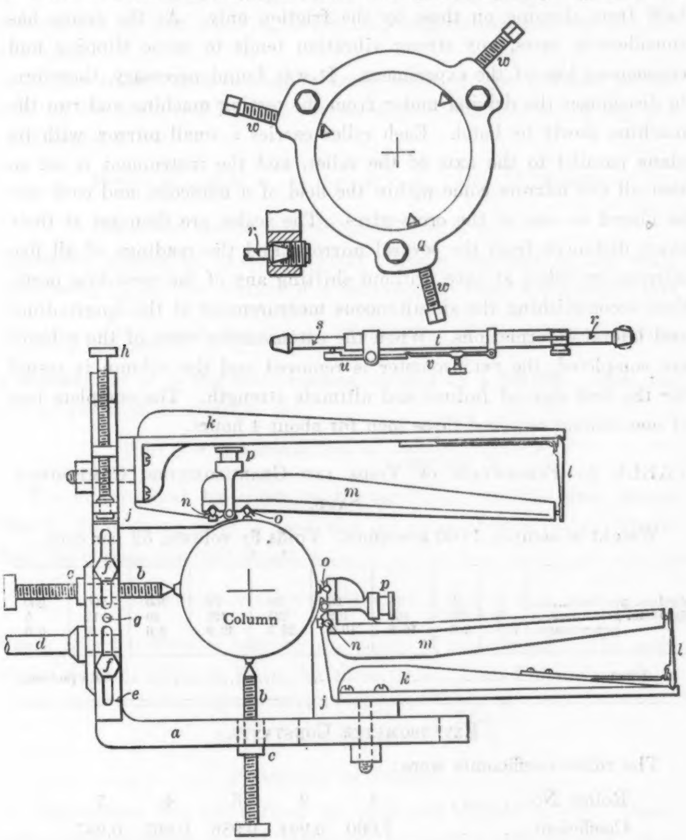

FIG. 2.

bearing against both its knife-edge seat on *k* and the roller. When all is adjusted, the screws, *b*, are made fast by the lock-nuts, *o*. so that all is rigidly connected to *a* except the arms, *m*, and the rollers. Any increase or decrease of diameter will cause *m* to rotate about *n*

and produce rotation of the roller. The average of the change shown on the two diameters is taken as the change in diameter of the column. The instrument when adjusted is free to move with the column, but none of its weight is carried by its bearing on the glass plates; it is held from slipping on these by the friction only. As the frame has considerable mass, any strong vibration tends to cause slipping and consequent loss of the experiment. It was found necessary, therefore, to disconnect the driving motor from the testing machine and run the machine slowly by hand. Each roller carries a small mirror, with its plane parallel to the axis of the roller, and the instrument is set so that all five mirrors come within the field of a telescope, and each can be placed on one of the cross-wires. The scales are then set at their exact distances from the several mirrors, and the readings of all five mirrors are taken at once, without shifting any of the recording parts, thus accomplishing the simultaneous measurement of the longitudinal and lateral deformations. When the extensometer tests of the column are completed, the extensometer is removed and the column is tested for the first sign of failure and ultimate strength. The complete test of one column required three men for about 4 hours.

TABLE 2.—PERCENTAGE OF VOIDS AND GRANULOMETRIC COMPOSITION OF SAND.

Weight of sample, 1 000 grammes. Voids by volume, 32 per cent.

									Passed.
Meshes, per inch......	4	6	16	20	30	74	100	200	200
Residue, grammes....	5	26	168	102	225	429	26	12	5
" percentage.	0.5	2.6	16.8	10.2	22.5	42.9	2.6	1.2	0.5

Total.. 99.8 per cent.

EXTENSOMETER CONSTANTS.

The roller coefficients were:

Roller No........	1	2	3	4	5
Coefficient........	1.000	0.994	0.986	0.965	0.987

Each reading from a roller is multiplied by its coefficient to give the corrected reading.

Referring to Fig. 2: From n to the end of m is 10 times the length n-o. The roller between m and l has a diameter, $2r = \frac{1}{10}$ in., approximately. R is the distance from the center of the roller to the scale,

taken as 100 in. for the transverse extensometer and 50 in. for the longitudinal one. s_1 is the distance traversed on the scale by the lateral mirror when a load is applied, corrected as above stated. s_2 is the same for the longitudinal mirror. λ_1 is the unit lateral deformation for any load. λ_2 is the unit longitudinal deformation for any load.

If the roller turns through a small angle, θ, the distance passed over on the scale is $2R\theta$. At the same time, the motion of the end of m past l is $2r\theta$.

$$\text{Motion of end of } m : s_1 :: r : R :: \frac{1}{20} : 100 :: 1 : 2\,000.$$

Motion of end of m is 10 times that of o.

Therefore, motion of $o : s_1 :: 1 : 20\,000$. Hence the unit on the scale used being $\frac{1}{50}$ in., the motion of o necessary to make a reading of this unit is $\frac{1}{1\,000\,000}$, and the change of diameter of the specimen having a diameter of D is the scale reading divided by 1 000 000. Thus the change per inch of diameter is:

$$\text{Unit lateral deformation} = \frac{s_1}{1\,000\,000\,D} = \lambda_1.$$

For longitudinal deformations, the conditions are the same, except that no multiplying lever is used, and the value of R is 50 in. Therefore the change per unit length is

$$\text{Unit longitudinal deformation} = \frac{s_2}{50\,000\,L} = \lambda_2,$$

where L is the distance between the points of attachment of the rings to the column, taken as 12 in. for these tests. Then Poisson's ratio is

$$m = \frac{\lambda_1}{\lambda_2} = \frac{1\,000\,000\,s_2\,D}{600\,000\,s_1} = \frac{5\,s_2\,D}{3\,s_1}$$

Procedure in Testing.

The column was taken from the tank and allowed to dry. Diameters were calipered in two directions at right angles to each other at the bottom, center, and top. Two circles with planes normal to the axis were struck, 12 in. apart, to locate the attachment of the rings for the longitudinal measurement. A third was similarly struck for the position of the points for the lateral measurement. On this last circle, the extremities of two diameters at right angles to each

other being located, small blocks of plate glass were cemented to the column with faces made parallel by pressing them to their bed between two parallel steel faces. When these were firmly set, the rings were attached and the column was set with top down on the bed of sand in the cast-iron cup. Cotton waste was tamped lightly around the column to retain the sand. The column, with cap in place, was then set in the testing machine in an upright position, the bottom end on a bed of sand in a cup like that used at the top, and a light load applied to hold it in place, care being taken to secure a full even bearing on the sand. The transverse extensometer was set and adjusted as heretofore described.

The zero reading was taken with a total load of 100 lb. on the column, and thereafter increasing loads were applied and readings taken at each load, up to a maximum, then decreasing loads down to zero were read. On a few columns the loads were run up and back a second time, but usually only one run was made. These readings being completed, the extensometer was removed and the column was tested to destruction.

Behavior Under Test.

The driving gears of the testing machine caused considerable vibration when in action, and in some cases caused the lateral extensometer to slip on the glass plates. The readings showed plainly when this took place, and several tests were necessarily discarded on this account. By running the machine slowly by hand, this trouble was minimized, and usable results were obtained. Usually, the readings were fairly uniform and consistent, but, occasionally, erratic results were found, such as one mirror starting off with negative readings for a few loads, and gradually returning to positive ones as the loads were increased. There was commonly a noticeable difference in the readings between the two lateral, or among the three longitudinal, mirrors, showing that the column underwent lateral deflection. However, when the readings were averaged and plotted, the results usually were very fair curves.

In applying the increasing loads, the weight was set at the proper mark and the load was run up until the beam rose, the reading then being taken. The beam was never held up by the column, but always dropped, showing progressive deformation.

FIG. 3.—APPEARANCE OF CERTAIN TYPICAL COLUMNS AFTER TEST.

In loading to destruction, considerable lateral deflection usually appeared, the column frequently approximating the characteristic curve of the slender column with fixed ends. Extreme care was taken to observe the first sign of failure, which was shown by a flaking of the surface cement, commonly near the center of length of the column. In the columns without reinforcement, failure was abrupt, only two or three giving any previous indication, and that at a very small percentage below the ultimate load. In those with spiral reinforcement, it was always gradual, giving ample warning and time for observation. The outer concrete separated from the core in considerable sections and fell away, leaving the spiral exposed; with increasing loads, deflection appeared, the concrete of the core began to force out the hooping, and finally the latter broke, the concrete flying out at the point of the break as though blown out by a small charge of explosive; the wire showed a clean tension break. In a few cases the column failed at or near the end, showing imperfect construction. These cases were discarded unless the values were fairly high, as the local failure prevented the column from developing its full strength.

Computation of Values.

The area of cross-section was computed for all plain columns as a circle having a diameter equal to the mean of the six measured, two each at top, center, and bottom. The unit load for values of s_1, s_2, e, and m, for both plain and reinforced columns, are based on the area thus found.

For reinforced columns, the ultimate unit strength and the unit load at the first sign of failure are based on the area of core enclosed by the spiral. The percentage of steel is based on the volume of steel and the volume of core. The percentage shown in Column 14 of Table 3 is the ratio of total load at first sign of failure to total ultimate load. The average lateral deformation is the average of the distances traversed by the lateral extensometer mirrors on the scale, in fiftieths of an inch; and, as the column diameter on which this measurement was taken was in all cases very near 4 in., this reading gives the increase in lateral dimension of the unit cube in units of $\frac{1}{4\,000\,000}$ in. Similarly, the longitudinal deformation is the average of the readings of the three longitudinal mirrors, and gives the longitudinal deformation of the unit cube in units of $\frac{1}{800\,000}$ in. The

value of Poisson's ratio would be approximately $\frac{4\,000\,000\, s_2}{600\,000\, s_1}$, or $\frac{20\, s_2}{3\, s_1}$, but, in computing this, the exact diameter was used instead of the approximation of 4 in., so that the coefficient varies slightly from $\frac{20}{3}$.

The value of $e = \frac{E \text{ of steel}}{E \text{ of concrete}}$, and is used because it is quite as readily computed and plotted as the E of concrete, and, for purposes of this discussion, is somewhat more useful. It is computed as follows:

E of steel is taken as 29 000 000;

f_c = stress on concrete, in pounds per square inch;

$$E_c = \frac{600\,000\, f_c}{s_2};$$

$$\frac{E_s}{E_c} = \frac{29\,000\,000\, s_2}{600\,000\, f_c} = \frac{145\, s_2}{3\, f_c}.$$

The 10-in. slide-rule was used wherever time could be saved thereby, except for values which could be taken directly from tables.

Figs. 4 to 9, inclusive, show typical curves of s_2, e, and m, plotted to uniform scale, for columns Nos. 38, 46, 51, 58, 61, and 63. The unit for the s_2 curves is $\frac{1}{600\,000}$ in., that for e and m is unity. The curves for any column in the series may be plotted from the data for s_2, e_1, and m, in Tables 5, 6, and 7. The curves on Figs. 4 to 9, however, show the effect of unloading the column, as well as loading, but the tables do not.

Comparison of Ultimate Strength.

The effect of the reinforcement on the ultimate strength is shown by the comparisons in Table 4. The columns are grouped as follows:

Group	1.	Reinforcement	0.00 per cent.
	2.	"	0.44 to 0.54 per cent.
	3.	"	0.67 to 0.86 "
	4.	"	1.16 to 1.35 "
	5.	"	1.41 to 1.69 "
	6.	"	2.02 to 2.32 "
	7.	"	4.10 per cent. (One column only.)

TABLE 3.—LIST OF COLUMNS TESTED, AND RESULTS OF TESTS OF ULTIMATE STRENGTH.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Column No.	Average diameter.	Mean area.	Age, in days.	Diameter of coil.	Diameter of wire.	Pitch of coil.	Tension.	Percentage.	First Sign.		Ultimate.		Percentage.
									Total.	Unit.	Total.	Unit.	
10	4.03	12.76	28	3.6	0.105	0.50	......	1.87	18 000	1 770	30 770	3 020	58
11	4.05	12.88	28	3.5	0.104	0.75	......	1.25	23 000	2 390	30 100	3 130	76
12	4.03	12.76	28	3.5	0.134	0.75	......	2.09	23 000	2 390	32 300	3 360	71
13	4.03	12.76	28	3.5	0.165	1.00	......	2.30	26 000	2 700	42 100	4 370	62
14	4.06	12.95	28	3.5	0.165	1.50	......	1.54	20 000	2 080	32 500	3 380	62
15	4.02	12.69	28	4.02	0.079	0.25	62	1.95	32 000	2 520	52 800	5 490	60
16	4.19	13.79	28	4.19	0.079	0.25	62	1.87	35 000	2 540	45 550	4 740	77
17	4.20	13.85	28	4.20	0.079	0.25	125	1.87	37 000	2 670	51 100	5 310	72
18	4.18	13.72	28	4.18	0.079	0.33	125	1.26	38 000	2 760	47 670	4 950	80
19	4.05	12.88	28	4.05	0.079	0.50	62	0.97	34 000	2 640	39 600	4 120	86
20	4.06	12.95	28	4.06	0·079	0.50	125	0.96	27 700	2 140	34 200	3 560	81
21	4.17	13.66	28	4.17	0.079	0.33	125	0.41	34 000	2 490	49 000	5 090	69
22	4.03	12.76	28	4.03	0.079	0.70	125	0.69	37 000	2 900	40 950	4 250	90
36	4.03	12.76	42	.....	......	......	......	......	14 120	1 108	....	......	100
37	4.05	12.88	41	......	......	......	......	......	12 260	950	....	......	100
38	4.02	12.69	63	......	......	......	......	......	22 660	1 790	....	......	100
39	4.05	12.88	40	......	......	......	......	......	14 560	1 130	....	......	100
41	4.05	12.88	40	......	......	......	......	......	10 500	815	11 640	905	90
42	4.03	12.76	58	......	......	......	......	......	21 980	1 720	....	......	100
43	4.07	13.01	73	......	......	......	......	......	16 600	1 275	....	......	100
44	4.05	12.88	74	......	......	......	......	......	17 840	1 385	....	......	100
45	4.03	12.76	268	......	......	......	......	......	34 980	2 740	....	......	100
46	4.03	12.76	296	......	......	......	......	......	37 700	2 955	....	......	100
47	4.05	12.88	288	......	......	......	......	......	38 000	2 955	....	......	100
48	4.04	12.82	414	......	......	......	......	......	45 800	3 570	....	......	100
49	4.02	12.69	71	......	......	......	......	......	29 890	2 345	....	......	100
50	4.05	12.88	65	......	......	......	......	......	....	......	....	......	......
50*a*	4.03	12.76	340	......	......	......	......	......	37 000	2 900	....	......	100
51	4.04	12.82	434	......	......	......	......	......	46 300	3 600	....	......	100
52	4.04	12.82	440	.....	......	......	......	......	35 000	2 725	....	......	100
53	4.02	12.69	295	......	......	......	......	......	39 570	3 120	....	......	100
54	4.05	12.88	441	......	......	......	......	......	40 700	3 160	....	......	100
56	4.04	12.82	58	3.50	0.079	0.33	......	1.68	25 000	2 600	40 590	4 220	61
57	4.02	12.69	59	3.56	0.079	0.33	......	1.65	24 260	2 430	39 000	3 940	62
58	4.03	12.76	61	......	......	......	......	......	22 420	1 760	....	......	100
59	4.05	12.88	67	3.50	0.105	0.75	......	1.31	30 600	3 180	40 250	4 184	76
60	4.04	12.82	311	3.62	0.079	0.75	......	0.72	32 000	3 100	38 200	3 700	83
61	4.04	12.82	77	......	......	......	......	......	29 450	2 300	....	......	100
62	4.03	12.76	434	......	......	......	......	......	36 350	2 850	....	......	100
63	4.05	12.88	395	3.69	0.105	0.75	......	1.24	27 500	2 575	37 500	2 910	73
64	4.03	12.76	435	3.38	0.079	0.75	......	0.67	37 500	3 180	41 500	3 520	90
65	4.05	12.88	316	3.62	0.105	0.75	......	1.26	37 500	3 630	42 160	4 085	88
66	4.03	12.76	74	3.62	0.079	0.75	......	0.72	19 570	1 895	29 000	2 810	67
67	4.02	12.69	328	......	......	......	......	......	31 300	2 470	....	......	100
68	4.03	12.76	321	3.69	0.105	0.75	......	1.24	31 400	2 940	36 570	3 420	85
69	4.03	12.76	323	3.62	0.079	0.75	......	0.72	24 700	2 390	31 350	3 040	78
70	4.04	12.82	393	3.37	0.079	0.25	......	2.32	27 000	3 020	53 690	6 000	50
71	4.05	12.88	317	3.56	0.062	0.75	......	0.45	27 120	2 720	27 450	2 770	98
72	4.03	12.76	388	3.62	0.079	0.75	......	0.72	31 000	3 000	33 500	3 246	92
73	4.04	12.82	428	3.62	0.105	0.75	......	1.26	46 500	4 500	48 600	4 710	95
74	4.06	12.95	370	3.62	0.062	0.75	....	0.44	47 000	4 550	47 800	4 630	98
75	4.04	12.82	300	3.56	0.062	0.62	......	0.54	34 000	3 400	40 500	4 090	83
76	4.03	12.76	376	......	......	......	......	......	27 250	2 140	28 000	2 200	97
77	4.03	12.76	277	3.62	0.105	0.62	......	1.51	38 000	3 680	42 700	4 140	89
78	4.04	12.82	281	3.56	0.062	0.62	......	0.54	19 000	1 900	31 300	3 160	60
79	4.06	12.95	416	......	......	......	......	.....	28 060	2 170	....	......	100
80	4.05	12.88	417	......	......	......	......	......	29 000	2 250	....	......	100
81	4.02	12.69	270	3.62	0.079	0.62	.....	0.86	34 000	3 290	36 250	3 520	93
82	4.04	12.82	413	3.62	0.062	0.62	......	0.52	33 500	3 240	33 700	3 265	99
84	4.03	12.78	42	......	.....	.....	......	......	....	......	....	......	......

TABLE 3.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Column No.	Average diameter.	Mean area.	Age, in days.	Diameter of coil.	Diameter of wire.	Pitch of coil.	Tension.	Percentage.	First Sign.		Ultimate.		Percentage.
									Total.	Unit.	Total.	Unit.	
85	4.05	12.91	42	......	......	......	......	......	......	......	......	......	......
87	4.03	12.78	35	3.37	0.105	0.25	......	4.10	27 600	3 090	71 300	7 965	39
89	4.04	12.84	35	3.37	0.079	0.25	......	2.81	21 600	2 420	49 900	5 580	43
90	4.04	12.80	35	3.37	0.079	0.25	......	2.81	......	......	......	......	......
92	4.04	12.80	34	3.37	0.062	0.25	......	1.41	21 500	2 410	24 000	2 680	89
93	4.03	12.76	34	3.37	0.062	0.25	......	1.41	......	......	......	......	......
94	4.03	12.76	35	3.37	0.062	0.25	......	1.41	26 500	2 970	35 270	3 940	75
95	4.03	12.78	35	3.37	0.105	0.50	......	2.02	......	......	......	......	......
96	4.03	12.74	35	3.37	0.105	0.50	......	2.02	22 600	2 530	42 700	4 770	53
97	4.03	12.76	56	3.37	0.105	0.50	......	2.02	33 300	3 730	51 580	5 760	64
98	4.04	12.81	31	3.37	0.079	0.50	......	1.16	19 140	2 140	31 500	3 520	61
99	4.03	12.77	31	3.37	0.079	0.50	......	1.16	......	......	......	......	......
100	4.04	12.80	31	3.37	0.079	0.50	......	1.16	......	......	......	......	......
101	4.03	12.74	29	3.37	0.062	0.50	......	0.71	16 000	1 790	22 000	2 460	73
102	4.04	12.80	29	3.37	0.062	0.50	......	0.71	......	......	......	......	......
104	4.04	12.79	35	3.37	0.105	0.75	......	1.35	17 000	1 900	24 500	2 740	69
105	4.05	12.88	49	3.37	0.105	0.75	......	1.35	27 000	3 020	40 000	4 470	67
106	4.04	12.80	44	3.37	0.105	0.75	......	1.35	32 600	3 650	34 800	3 890	93
107	4.03	12.74	35	3.37	0.079	0.75	......	0.78	22 500	2 520	28 000	3 130	80
110	4.03	12.75	35	3.37	0.062	0.75	......	0.47	21 000	2 350	23 700	2 650	89
111	4.04	12.78	35	3.37	0.062	0.75	......	0.47	......	......	......	......	......
113	4.04	12.79	35	......	......	......	......	......	17 200	1 350	......	......	100
114	4.04	12.80	35	......	......	......	......	......	......	......	......	......	......
115	4.04	12.80	35	......	......	......	......	......	......	......	......	......	......

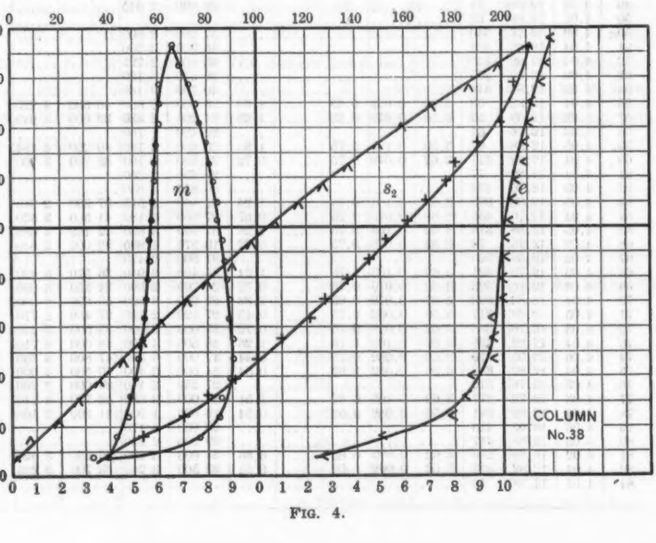

FIG. 4.

Plotting the average ultimate strength of each group, as shown on Fig. 10, gives four points for Group 1 and two for each of the others, except Group 7, which falls entirely off the diagram. The curve of Group 1 shows a consistent increase of strength with age, approximating in form to that of the age-strength curve of neat cement in tension, though the early increase is relatively less. For Groups 2 to 6 the ultimate strength is shown in full lines and the first sign of failure in broken lines.

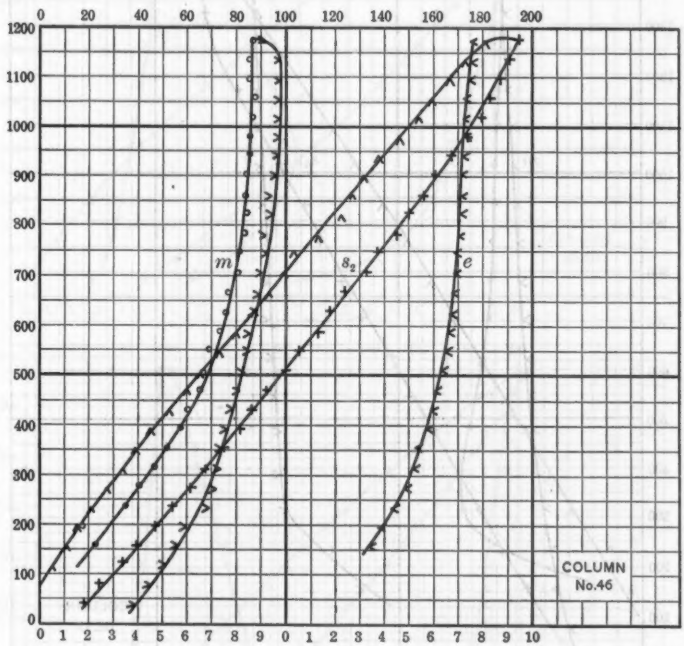

FIG. 5.

There are some irregularities in the relations of these lines, which are probably due to accidental causes, except the reversal of position of 2 and 3, which will be discussed later. However, all groups show an increase of strength with age, and generally with increasing quantities of reinforcement. The load for first sign of failure shows considerable irregularity in the long-time tests in Groups 5 and 6, but, as each of

these sub-groups contains results of one column only, such irregularity is plainly accidental.

Although the relation of the first sign of failure to ultimate strength is brought out more clearly by another diagram, this plotting is given to show the relations within the individual groups. The lack of har-

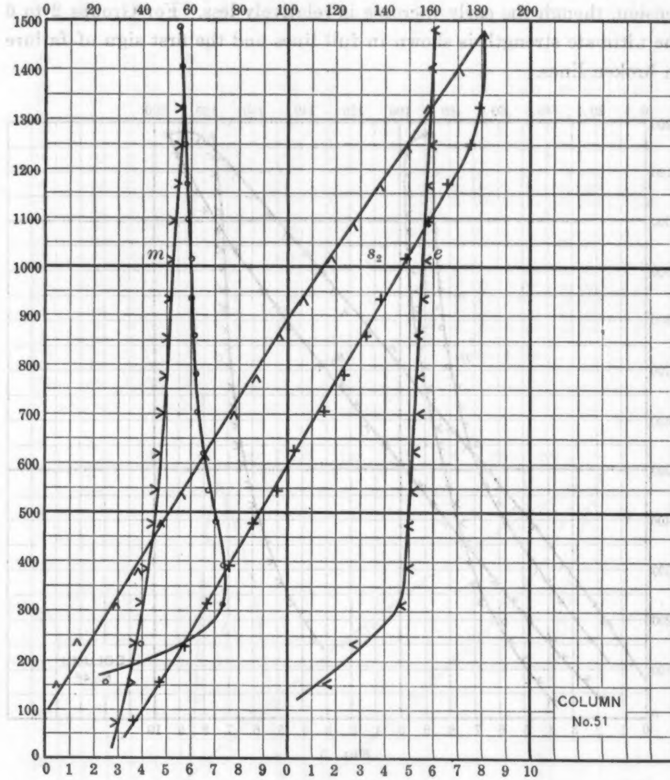

FIG. 6.

mony as the number of tests decreases, and the danger of generalizing from a few tests, are here clearly indicated.

The percentages of Column 14 of Table 3 are plotted on Fig. 11, using percentages of steel as abscissas and the values of Column 14 as ordinates. Here the short-time points, with the exception of that for

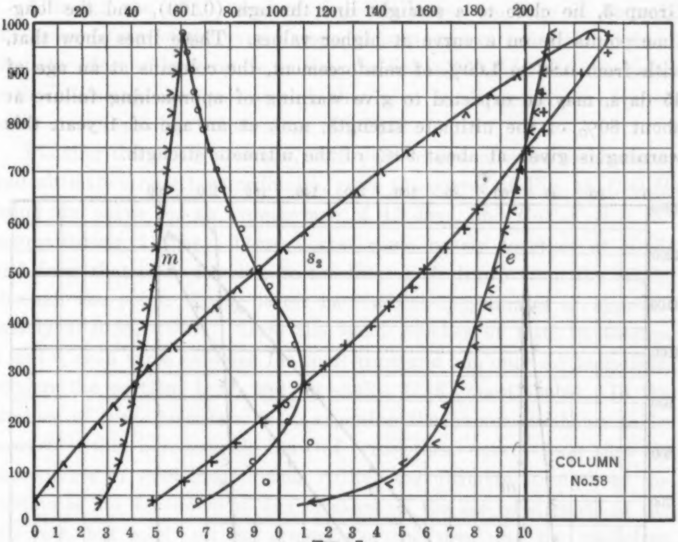
0
20
40
60
80
100
120
140
160
180
200
1000
900
800
700
600
500
400
300
200
100
0
m
s_2
e
COLUMN
No.58
0 1 2 3 4 5 6 7 8 9 0 1 2 3 4 5 6 7 8 9 10

FIG. 7.

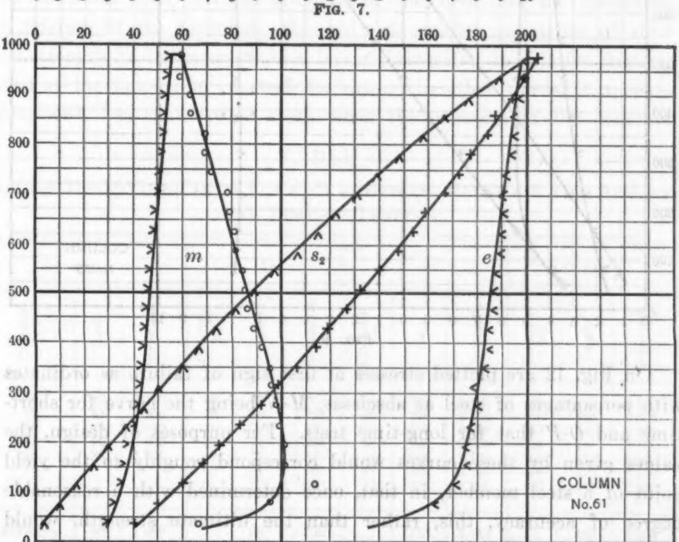
0
20
40
60
80
100
120
140
160
180
200
1000
900
800
700
600
500
400
300
200
100
0
m
s_2
e
COLUMN
No.61
0 1 2 3 4 5 6 7 8 9 0 1 2 3 4 5 6 7 8 9 10

FIG. 8.

Group 3, lie close to a straight line through (0.100), and the long-time points lie on a curve at higher values. These lines show that, with from 0.80 to 1.60% of reinforcement, the columns at an age of 45 days, may be expected to give warning of approaching failure at about 80% of the ultimate strength, and, at an age of 1 year, this warning is given at about 90% of the ultimate strength.

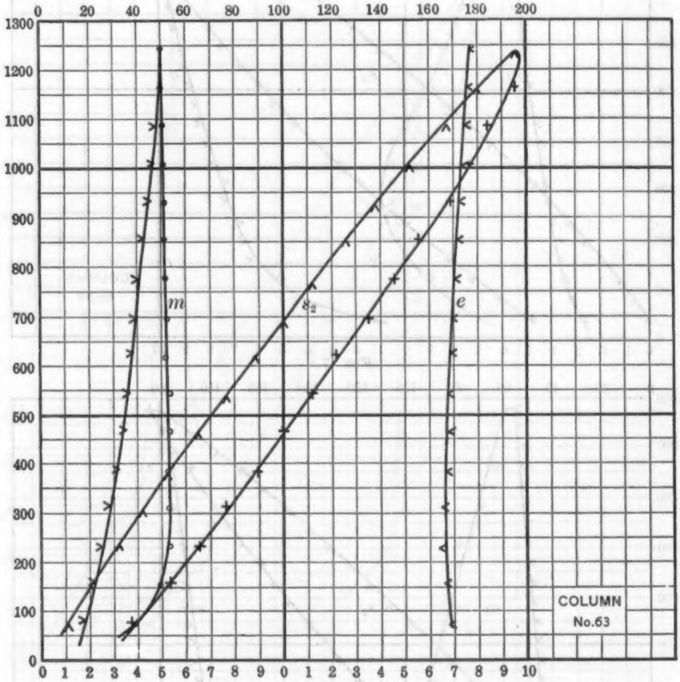

FIG. 9.

On Fig. 12 are plotted stresses at first sign of failure as ordinates with percentages of steel as abscissas, *M-N* being the curve for short-time and *O-P* that for long-time tests. For purposes of design, the values given by these curves would correspond roughly to the yield point of a steel member, in that, once determined with a reasonable degree of accuracy, this, rather than the ultimate strength, would control the allowable limit of working stress.

Although the small number of points leaves much to be desired, the general form of the curves seems to point to two conclusions: first, that the effect of hooping, in raising the first sign of failure, decreases with increasing age; and second, that even small quantities of steel exert an appreciable influence on this limit.

Plotting the group averages, with percentages of steel as abscissas and ultimate strength as ordinates, gives the curves of Fig. 13, *C-G-H* being the curve for an average age of 45 days, and *E-G′-K* that for approximately 1 year. These curves come nearly together at *G-G′*, and from that point lie close to the line, *A-B*, drawn from the origin through the point, *G*, the latter having the co-ordinates of approximately (0.0158, 4 000). Here the small number of tests in Groups 5 and 6, each three columns for short-time and one only for long-time, renders the portion, *G-H*, and especially, *G′-K*, questionable. In the absence of more complete data, assuming the curves as shown to be correct, within a reasonable limit of error, these curves show that for percentages of steel greater than 0.0158, the ultimate strength of the column is not dependent in any measure on the age or strength of the concrete, but solely on the strength of the steel and the coefficient of friction of the aggregate, though with smaller quantities of steel the strength of the concrete is a factor, increasing in relative importance as the proportion of steel decreases. In other words, it appears that, with less than 0.0158 steel, when the cohesion of the concrete

TABLE 4.

GROUP 1.—DIVIDED INTO FOUR SUB-GROUPS, BASED ON THE AGE AT THE TIME OF TESTING.

(a)			(b)			(c)			(d)		
Column No.	Age.	Strength.	Column No.	Age.	Strength.	Column No.	Age.	Strength.	Column No.	Age.	Strength.
36	42	1 110	38	63	1 790	45	268	2 740	48	414	3 570
37	41	950	42	58	1 720	46	296	2 955	51	434	3 600
39	40	1 130	43	73	1 275	47	288	2 955	52	440	2 725
41	40	905	44	74	1 385	50*a*	340	2 900	54	441	3 160
113	35	1 350	49	71	2 345	53	295	3 120	62	434	2 850
			58	61	1 760	67	323	2 470	79	416	2 170
			61	77	2 300	76	376	2 200	80	417	2 250
Averages..	40	1 089	..	68	1 796	..	313	2 763	..	428	2 903

TABLE 4.—(*Continued*).

GROUPS 2 to 6 ARE EACH DIVIDED INTO TWO SUB-GROUPS.

Column No.	Age.	Ultimate strength.	Percentage of steel.	First sign.	Percentage of ultimate.	Column No.	Age.	Ultimate strength.	Percentage of steel.	First sign.	Percentage of ultimate.
GROUP 2.											
110	35	2 650	0.47	2 380	89	71	317	2 773	0.45	2 720	99
						74	370	4 632	0.44	4 550	98
						75	300	4 091	0.54	3 410	84
						78	281	3 160	0.54	1 910	61
						82	413	3 265	0.52	3 240	99
Averages..	..	....	....	....	..	..	336	3 584	0.50	3 166	88
GROUP 3.											
66	74	2 810	0.72	1 890	67	60	311	3 702	0.72	3 100	84
101	29	2 460	0.71	1 790	73	64	435	3 520	0.67	3 170	90
107	35	3 130	0.78	2 520	80	69	323	3 040	0.72	2 390	70
						72	388	3 246	0.72	3 000	93
						81	270	3 520	0.86	3 300	94
Averages..	46	2 800	0.74	2 067	72	..	345	3 406	0.74	2 992	88
GROUP 4.											
59	67	4 184	1.31	3 180	76	63	395	3 520	1.24	2 570	73
98	31	3 520	1.16	2 130	61	65	316	4 085	1.26	3 640	89
104	35	2 740	1.35	1 900	69	68	321	3 420	1.24	2 940	86
105	49	4 470	1.35	3 020	67	73	428	4 710	1.26	4 500	96
106	44	3 890	1.35	3 640	93						
Averages..	45	3 761	1.30	2 774	73	..	365	3 934	1.25	3 412	86
GROUP 5.											
56	58	4 219	1.68	2 600	62	77	277	4 138	1.51	3 680	89
57	59	3 939	1.65	2 430	62						
94	35	3 940	1.41	2 960	75						
Averages..	51	4 033	1.58	2 663	66						
GROUP 6.											
89	35	5 580	2.31	2 410	43	70	393	6 000	2.32	3 020	50
96	35	4 770	2.02	2 520	53						
97	56	5 760	2.02	3 720	64						
Averages..	42	5 370	2.11	2 883	53						
GROUP 7.											
87	35	7 965	4.10	3 080	39						

disappears, the steel does not suffice to carry the load by its lateral hold on the granular, non-coherent mass, but that with the failure of cohesion of the concrete occurs the failure of the column; and, with a larger quantity of steel than this, the failure of cohesion in the

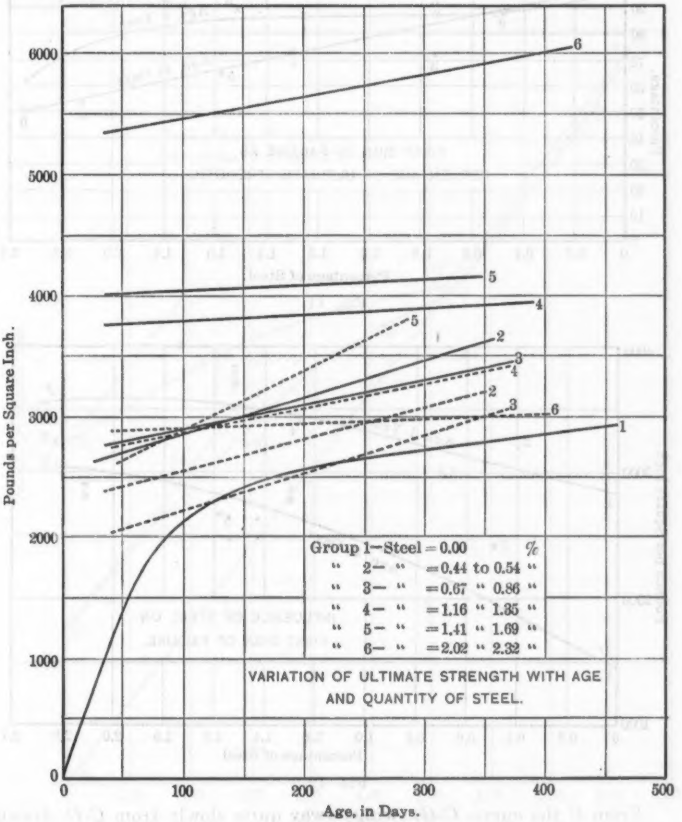

FIG. 10.

concrete does not produce failure of the column, as the steel acting on the core suffices to support the load.

Attention is here directed to the fact that this limit of 0.0158 is based on the volume ratio of steel to concrete, on wire having a yield

point and ultimate strength as shown in the table, and on a broken-stone concrete. With a weaker steel, or with aggregate material having a different coefficient of friction, this limit would be changed.

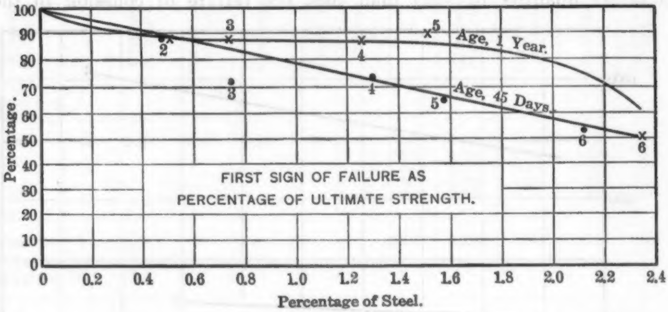

FIG. 11.

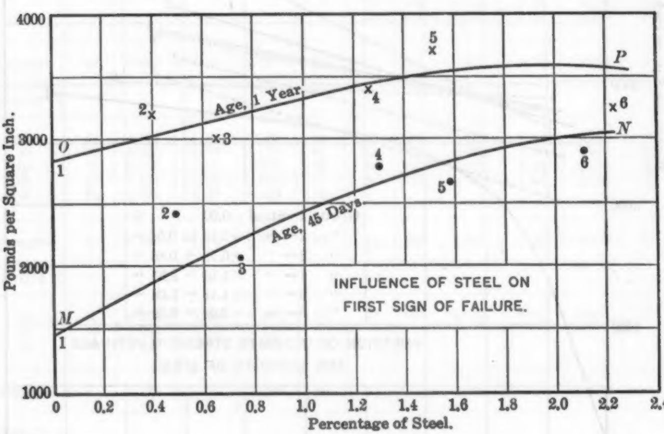

FIG. 12.

From C the curve, C-G, swings away quite slowly from C-D, drawn parallel to A-G, showing a more rapid breaking down of the cement bond as the load increases; though E-G draws away from the similar parallel, E-F, much more rapidly. Both, however, indicate clearly that the reinforcement is effective in raising the ultimate strength, even with small proportions of steel.

The ordinate to the curve at any point consists of two portions: that below *A-B,* due to the wire confining the granular mass, and that above *A-B,* due to the cohesion of the concrete; or, perhaps better, that below *A-B,* due to the reinforcing steel, and that above due to the cement bond.

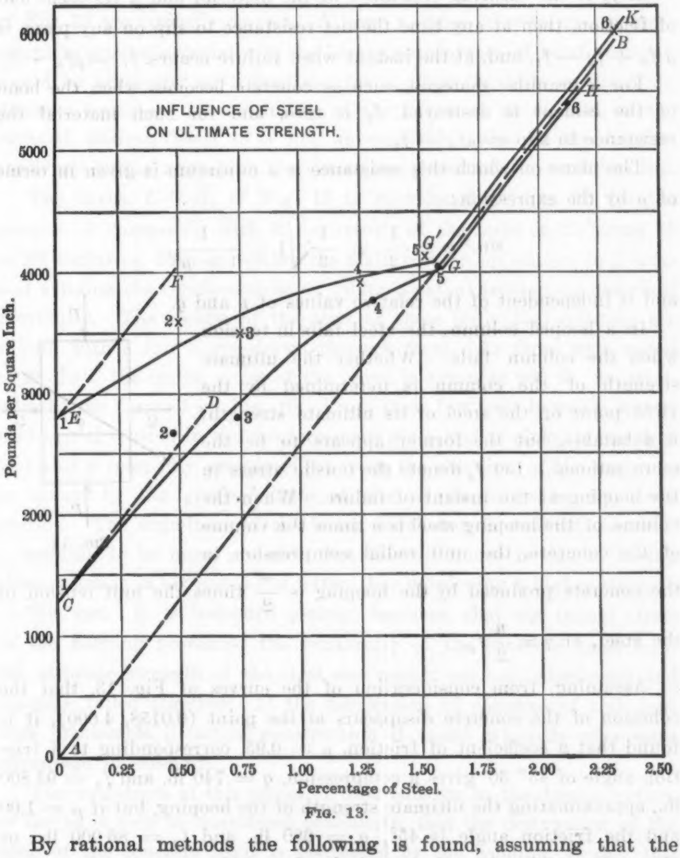

FIG. 13.

By rational methods the following is found, assuming that the column fails by shear on an oblique plane: Given a particle acted on by the forces, *P* and *Q,* at right angles to each other, producing stresses *p* and *q* on their normal planes (*p* being greater than *q*),

the resultant normal and tangential stresses on a plane, M-N, making an angle, ϕ, with the direction of Q will be

$$f_n = p \cos.^2 \phi + q \sin.^2 \phi$$
$$f_t = (p - q) \cos. \phi \sin. \phi$$

If f_z is the shearing resistance of the material and μ its coefficient of friction, then at any time the net resistance to slip on any plane is $\mu f_n + f_z - f_t$, and, at the instant when failure occurs, $f_t = \mu f_n + f_z$.

For a granular material, such as concrete becomes when the bond of the cement is destroyed, f_z is zero, and for such material the resistance to slip $= \mu f_n - f_t$.

The plane on which this resistance is a minimum is given in terms of μ by the expression:

$$\sin.^2 \phi = \frac{1}{2} \pm \frac{1}{2}\sqrt{1 - \frac{1}{1+\mu^2}},$$

and is independent of the relative values of p and q.

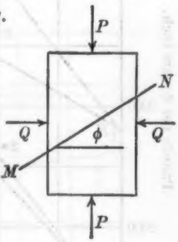

FIG. 14.

In a hooped column, the steel fails in tension when the column fails. Whether the ultimate strength of the column is determined by the yield point of the steel or its ultimate strength, is debatable, but the former appears to be the more rational. Let f_s denote the tensile stress in the hooping at the instant of failure. When the volume of the hooping steel is n times the volume of the concrete, the unit radial compression in the concrete produced by the hooping is $\frac{n}{2}$ times the unit tension in the steel, or $q = \frac{n}{2} f_s$.

Assuming, from consideration of the curves of Fig. 13, that the cohesion of the concrete disappears at the point (0.0158, 4 000), it is found that a coefficient of friction, $\mu = 0.95$, corresponding to a friction angle of 43° 30′ gives a compression, $q = 740$ lb. and $f_s = 93\,800$ lb., approximating the ultimate strength of the hooping, but if $\mu = 1.00$ and the friction angle is 45°, $q = 680$ lb. and $f_s = 86\,000$ lb., or about the yield point of the spiral. It would seem, therefore, that, with a steel having a relatively high yield point, the question whether ultimate strength or yield point determines the strength of the column

is not an important one, because so small a variation in the angle of friction corresponds to so large a range of stress in the steel.

This method of treatment takes no account of flexure in the column, and, therefore, is applicable only to short columns.

The writers do not consider that the tests prove that the curves of Fig. 13 are typically correct. A large number of tests, giving more complete groups, especially in the higher percentages of reinforcement, would be necessary for a final conclusion; but the tests as given indicate a break in the continuity of the curve, as at *G*, and, as the rational analysis leads to a not unreasonable value of the friction coefficient in the aggregate, the curves are presented for consideration.

The curve, *C-G-H*, of Fig. 13 is reproduced in Fig. 15 for the purpose of comparing with it the results of the tests on Columns 15 to 22, inclusive, built by setting the plain column on centers in a lathe and winding the reinforcement on under constant tension, as described previously. The results of the tests of these individual columns are shown, with a fair curve drawn through them. As these were tested at 28 days, the initial point of the curve is taken at 900 lb., as determined from Curve 1 on Fig. 10. From these few tests it would appear that initial tension in the reinforcement gives higher ultimate strength and also a relatively greater effect from the cohesion of the concrete, as shown by the greater angle of rise of the curve in its lower portion. The soundness of the latter conclusion is doubted, however, it seeming to be more probable that this difference is due to curing these columns in air instead of water, thus developing a higher strength at this age. It is indicated plainly, however, that the initial stress in the hooping prevented the weakening of the concrete bond until the ultimate strength of the steel was nearly reached, when steel and concrete failed together. If the fabrication of columns of this type were practicable in actual construction it would evidently offer some advantage in strength.

The usefulness of this group of tests consists in the demonstration, so far as it may be considered conclusive, that the destruction of the bond of the concrete itself is restrained by the hooping, so that this destruction becomes a gradual process, instead of the abrupt one which occurs in the plain column, and that this restraining effect is produced, even with small percentages of steel.

On Fig. 15 are plotted also the points for Columns 10 to 14, inclusive, marked *z*; also from tests* by A. N. Talbot, M. Am. Soc. C. E., Columns 171, 172, 173, 181, 182, 183, reinforced with high-carbon steel, and marked *T*, and Columns 176, 177, 178, 186, 187, reinforced

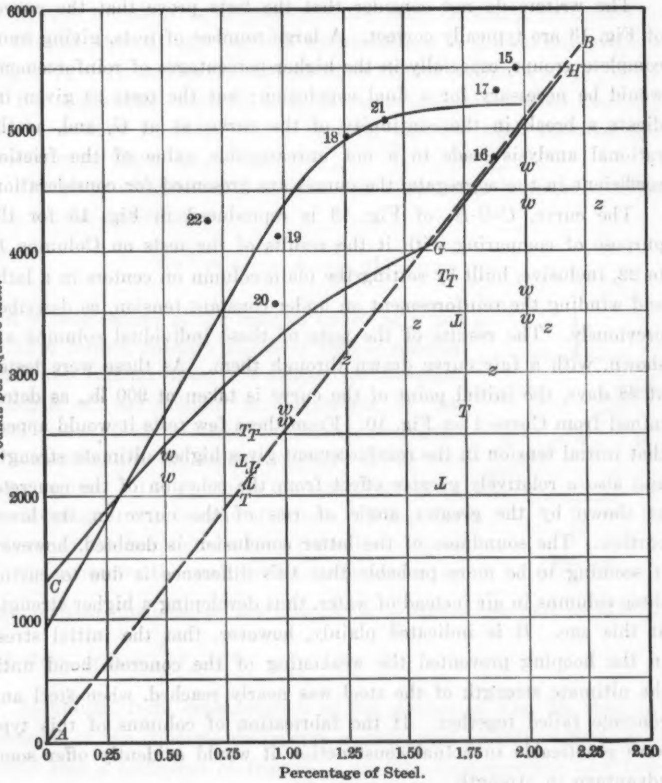

FIG. 15.

with mild steel and marked *L*; also from tests † by Professor M. O. Withey, Columns *C*1, *C*2, *C*3, *C*4, *H*1, *H*2, *L*1, *L*2, marked *w*. All the columns of this group are reinforced with hooping prepared by commercial firms, and differ only in minor details. Compared with

* Bulletin No. 20, Illinois Experiment Station.

† Bulletin No. 466, University of Wisconsin.

Columns 15 to 115 of this series, they differ first in having longitudinal or spacing members relatively much heavier, and second in the use of a heavier hooping wire spaced on a higher pitch.

The fact, noted by Considère, that a low-pitch spiral is more effective than one of high pitch is brought out clearly here. These points lie in a fairly close group, but all are well below the curve, *C-G-H*.

There is no reason to believe that these lower values are due to inferior concrete, nor, on account of their consistent grouping, can it be argued that they are due to differences in conditions of the tests, unless because of the fact that these were cured in air and the *C-G-H* columns were cured in water. A consideration of Fig. 10, however, shows the line of Group 2, with 0.44 to 0.54% of steel, lying above that of Group 3, with 0.67 to 0.86 per cent. It happens that all the columns of Group 2 were reinforced with the 0.061 wire, and that Groups 3, 4, and 5, have each only one column with this fine wire, all other reinforcement being heavier. The writers, therefore, believe that the points under discussion lie below *C-G-H,* because of the fact that the spiral is of coarser wire and consequently higher pitch.

Values of s_2.—The stress-strain curves show a permanent deformation which is greater in the short-time tests. This is shown by arranging in two groups, the first including all columns from 84 to 115, all of which were loaded under the extensometer to 700 lb. per sq. in., and all at approximately the same age. These show on the 100-lb. line a length of 53 units (of $\frac{1}{500\,000}$ in.) between the ascending and descending curves. The second group includes Columns 46, 48, 51, 52, 53, 54, 60, 62, 64, 67, 69, 71, 73, 74, 75, 79, 80, 81, and 82, all but four of which were loaded to 1 400 lb., and all above 1 150 lb. have an average age of 352 days and show on the 100-lb. line a length of 35 units.

In the majority of cases these curves show a satisfactory uniformity, but some exceptions appear, as Column 72, tested at 388 days, which shows an abnormal lack of resilience, the load dropping from 1 400 to 1 000 lb. before recovery began.

For examining these results to determine the influence of the hooping on them, the columns are grouped in Table 5 in the same manner as for ultimate strength. The strains here are read off from the plotted curves, instead of being taken from the tables.

The average values of these strains, using ages in days as ordinates and strains as abscissas, are plotted on the diagram, Fig. 16, where

the numerals show the group numbers. Separate curves are shown for stresses of 400 to 900 lb. per sq. in. Here appears the unexpected condition that the plain columns show less strain than those which are reinforced. Before beginning the tests the writers had expected that, at least in the higher stresses, the columns would show, both in the s_2 and the *m*, just the opposite. If the strain in the reinforced columns had been less than in the plain ones, or if no difference between the two had appeared, the explanation would be simple; but, that the presence of a spiral of steel should be accompanied by a greater deformation

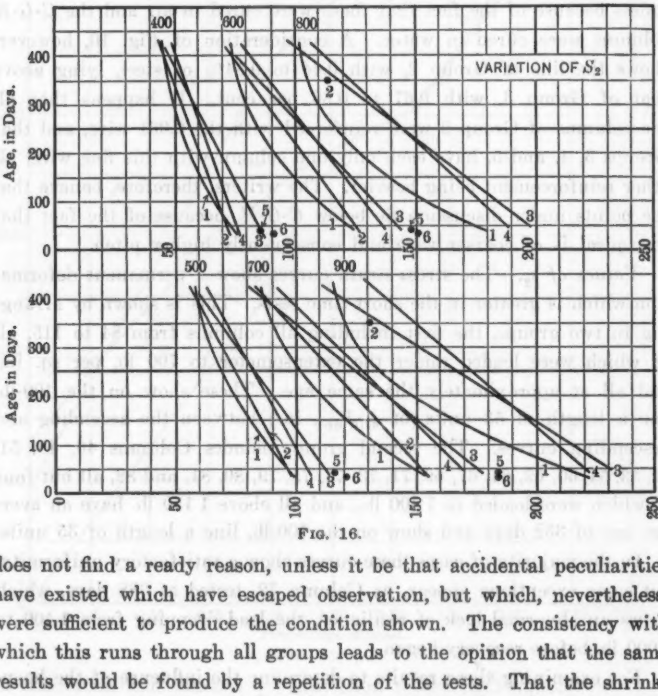

FIG. 16.

does not find a ready reason, unless it be that accidental peculiarities have existed which have escaped observation, but which, nevertheless, were sufficient to produce the condition shown. The consistency with which this runs through all groups leads to the opinion that the same results would be found by a repetition of the tests. That the shrinkage of the concrete, causing initial compression in the steel, might be the cause is untenable, first, because the concrete was immersed in water during the entire aging period, and the shrinkage, therefore, would not occur so as to produce this result; and second, because if the steel spiral were under initial compression one would expect to find

TABLE 5.—Values of s_2.

Group 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.......	42	79	108	135	163	......	......	......	......	......	......	......
114.......	35	68	91	115	144	......	......	......	......	......	......	......
115.......	35	65	90	115	142	......	......	......	......	......	......	......
Average....	37	71	96	122	150	......	......	......	......	......	......	......
50.......	65	70	91	113	136	161	......	......	......	......	......	......
38.......	63	81	104	128	158	188	......	......	......	......	......	......
42.......	58	68	90	113	135	160	......	......	......	......	......	......
43.......	73	85	112	143	176	213	251	......	......	......	......	......
44.......	74	78	103	131	160	192	227	......	......	......	......	......
58.......	61	67	90	114	140	168	197	......	......	......	......	......
61.......	77	68	88	108	130	153	177	......	......	......	......	......
Average....	67	74	97	121	148	176	213	......	......	......	......	......
46.......	296	47	65	83	98	114	132	149	167	......	......	......
47.......	288	72	89	106	122	138	153	......	......	......	......	......
53.......	295	42	54	66	79	92	105	119	132	......	......	......
67.......	322	40	53	67	82	97	112	128	143	160	177	196
Average....	300	50	65	80	95	110	125	132	147	160	177	196
48.......	414	36	48	59	71	84	97	110	123	136	148	161
51.......	434	38	51	63	75	88	100	114	128	142	156	170
52.......	440	44	57	71	85	99	113	128	142	156	170	184
54.......	441	43	54	66	78	90	101	114	128	142	156	170
62.......	434	53	68	82	96	112	128	145	161	178	194	212
79.......	416	45	60	75	91	107	123	140	158	178	198	......
80.......	417	59	75	94	112	130	150	170	192	215	240	264
Average....	428	45	59	73	86	101	116	132	146	164	180	193

Group 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
111....	35	0.50	73	99	127	155	......	......	......	......	......	......	......
71....	317	0.45	51	64	78	92	106	121	137	154	172	190	211
74....	370	0.40	62	80	99	119	138	157	175	192	210	227	......
75....	300	0.54	38	54	70	86	103	120	139	159	181	205	230
82....	413	0.58	48	61	76	91	108	125	143	162	182	202	223
Average.	350	0.49	50	65	81	97	114	131	148	167	186	206	221

TABLE 5.—VALUES OF s_2.—(*Continued*).

GROUP 3.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
66....	74	0.72	83	107	133	161	191	225	......	......	......	......	......
101....	29	0.71	89	120	153	190	......	......	......	......	......	......	......
107....	35	0.78	83	113	147	179	......	......	......	......	......	......	......
Average.	46	0.74	85	113	144	177	191	225	......	......	......	......	......
60....	311	0.72	66	81	96	111	127	144	162	181	200	220	241
64....	435	0.67	44	59	74	89	105	122	140	158	177	198	218
69....	323	0.72	53	70	86	104	123	143	164	186	208	233	260
72....	388	0.72	57	73	89	105	123	141	160	181	205	230	259
81....	270	0.86	48	66	83	100	120	140	160	181	......	......	......
Average.	345	0.74	54	70	86	102	120	138	157	177	197	220	244
GROUP 4.													
59....	67	1.31	76	101	128	157	186	219	256	......	......	......	......
99....	31	1.16	76	110	146	183	......	......	......	......	......	......	......
Average.	49	1.23	76	105	137	170	186	219	256	......	......	......	......
68....	395	1.24	55	70	86	101	117	133	150	169	188	......	......
73....	428	1.26	46	60	74	89	104	120	135	152	169	188	206
Average.	411	1.25	50	65	80	95	110	126	142	160	178	188	206
GROUP 5.													
56....	58	1.69	84	115	148	184	227	......	......	......	......	......	......
92....	34	1.41	90	121	156	193	......	......	......	......	......	......	......
93....	34	1.41	89	120	153	187	......	......	......	......	......	......	......
94....	35	1.41	83	112	142	175	......	......	......	......	......	......	......
105....	35	1.35	91	125	161	202	......	......	......	......	......	......	......
106....	35	1.35	77	104	133	164	......	......	......	......	......	......	......
Average.	38	1.44	86	116	149	184	227	......	......	......	......	......	......
GROUP 6.													
89....	85	2.30	90	118	144	172	......	......	......	......	......	......	......
95....	85	2.04	94	124	160	199	......	......	......	......	......	......	......
96....	85	2.02	88	120	153	180	......	......	......	......	......	......	......
Average.	85	2.12	91	121	152	184	......	......	......	......	......	......	......

m, at short time, less for reinforced than for plain columns, whereas the curves for m show precisely the opposite. The only explanation which the writers can suggest is that the presence of the reinforcement, with its rather closely spaced wires, may have prevented in a measure the thorough elimination of bubbles and small voids, thus rendering the concrete as a whole less dense. Moreover, Professor Talbot, has discussed* the same condition, which he found to exist in his experiments, and on which he places the same interpretation.

These curves indicate that, within the limits of ordinary working stresses, spiral reinforcement does not noticeably lend assistance to the concrete in carrying the load, but that, if such assistance is developed, it is at stresses well above the ordinary working limits.

Values of e.—The values of e have been computed and plotted for the ascending curves only. In Table 6 the columns are grouped as before, and the values are as read from the plotted curves.

The average values from Table 6 are plotted on Fig. 17, the ages, in days, being the ordinates and the values of e the abscissas. The curves show a close agreement for both plain and reinforced columns at 425 days, and separate more widely with decreasing age and increasing stress. As these are derived directly from s_2, they show the same general relations between the plain and reinforced columns, and the comments made regarding the stress-strain curves apply with equal force here. The evidence of these curves is decidedly against the use of values of e greater than 10 for computing the strength of reinforced columns. It runs above that value only for the combinations of short time with high stresses, a combination to be avoided in practice as much as possible.

Values of m.—m is the ratio of longitudinal to lateral deformation, the two being opposite in kind when the member is stressed in one direction only, and its value, m, in terms of the observed quantities, becomes

$$m = \frac{\lambda_2}{\lambda_1} = \frac{5\,D\,s_2}{3\,s_1}.$$

This involves one more observed quantity, s_1, which is more difficult to measure, not only because it is very small, but also because of the likelihood of the instrument slipping. The curves show less

* Bulletin No. 20, Illinois Engineering Experiment Station.

TABLE 6.—VALUES OF e.

GROUP 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.......	42	9.4	10.2	10.8	11.2	......	......	......	......	......	......	......
114.......	35	8.3	8.9	9.4	9.8	......	......	......	......	......	......	......
115.......	35	7.8	8.6	9.3	9.9	......	......	......	......	......	......	......
Average....	37	8.5	9.2	9.8	10.3	......	......	......	......	......	......	......
38.......	63	9.8	10.1	10.4	10.8	11.5	......	......	......	......	......	......
42.......	58	8.3	8.8	9.2	9.4	9.6	......	......	......	......	......	......
43.......	73	10.4	10.9	11.4	12.0	12.7	13.3	......	......	......	......	......
44.......	74	9.6	10.1	10.5	11.0	11.5	12.2	......	......	......	......	......
50.......	65	8.4	8.8	9.1	9.4	9.8	......	......	......	......	......	......
58.......	61	8.0	8.6	9.2	9.7	10.2	10.7	......	......	......	......	......
61.......	77	8.2	8.5	8.8	9.0	9.3	9.6	......	......	......	......	......
Average....	67	8.9	9.4	9.8	10.2	10.7	11.4	......	......	......	......	......
46.......	296	5.7	6.2	6.6	6.9	7.0	7.1	7.2	7.4	......	......	......
47.......	288	8.7	8.6	8.5	8.4	8.3	8.2	......	......	......	......	......
53.......	295	5.1	5.2	5.3	5.5	5.6	5.7	5.8	5.9	......	......	......
67.......	323	4.9	5.1	5.3	5.5	5.7	5.9	6.1	6.3	6.4	6.5	6.6
Average....	300	6.1	6.3	6.4	6.6	6.7	6.7	6.4	6.5	6.4	6.5	6.6
48.......	414	4.3	4.5	4.8	5.0	5.1	5.2	5.4	5.5	5.5	5.5	5.5
51.......	434	4.9	5.0	5.1	5.2	5.3	5.4	5.5	5.7	5.8	5.9	6.0
52.......	440	5.4	5.7	5.8	5.9	6.0	6.0	6.1	6.1	6.2	6.2	6.3
54.......	441	5.0	5.1	5.2	5.3	5.4	5.5	5.5	5.6	5.7	5.8	5.9
62.......	434	6.4	6.5	6.6	6.7	6.8	6.9	7.0	7.1	7.1	7.2	7.3
79.......	416	5.4	5.7	6.0	6.2	6.4	6.6	6.8	7.0	7.2	7.4	7.5
80.......	417	7.0	7.2	7.4	7.6	7.8	8.0	8.2	8.4	8.6	8.8	9.0
Average....	428	5.5	5.7	5.8	6.0	6.1	6.2	6.4	6.5	6.6	6.7	6.8

GROUP 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100
111.........	35	0.47	8.9	9.6	10.2	10.8	......	......	......	......
71.........	317	0.45	6.1	6.2	6.3	6.4	6.5	6.5	6.6	6.7
74.........	370	0.44	7.7	7.9	8.1	8.2	8.3	8.3	8.3	8.3
75.........	300	0.54	4.6	5.1	5.5	5.9	6.2	6.4	6.7	7.0
82.........	413	0.52	5.8	6.0	6.2	6.4	6.6	6.8	7.0	7.1
Average......	350	0.47	6.1	6.3	6.5	6.7	6.9	7.0	7.1	7.3

TABLE 6.—(*Continued*).

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100
GROUP 3.										
66	74	0.72	10.1	10.4	10.7	11.0	11.4	12.0	......	......
101	29	0.71	10.7	11.5	12.3	13.0	......	......	......	......
107	35	0.78	10.1	11.0	11.8	12.2	......	......	......	......
Average	46	0.74	10.3	11.0	11.6	12.1	11.4	12.0	......	......
60	311	0.72	8.1	7.9	7.8	7.8	7.8	7.8	7.9	8.0
64	435	0.67	5.2	5.6	5.9	6.1	6.3	6.5	6.7	7.0
69	323	0.72	6.5	6.9	7.1	7.3	7.4	7.7	8.0	8.2
72	388	0.80	6.8	7.0	7.1	7.3	7.4	7.6	7.8	8.0
81	270	0.86	5.7	6.1	6.6	6.9	7.2	7.4	7.5	7.8
Average	344	0.74	6.5	6.7	6.9	7.1	7.2	7.4	7.6	7.8
GROUP 4.										
59	67	1.31	9.2	9.8	10.4	10.9	11.3	11.8	12.2	......
99	31	1.16	9.1	10.5	11.8	12.8	......	......	......	......
Average	49	1.23	9.1	10.1	11.1	11.8	11.3	11.8	12.2	......
63	395	1.24	6.6	6.7	6.8	6.9	7.0	7.1	7.3	7.4
78	428	1.26	5.5	5.8	6.0	6.2	6.3	6.5	6.6	6.7
Average	411	1.25	6.0	6.1	6.4	6.5	6.6	6.8	6.9	7.0
GROUP 5.										
56	58	1.69	10.1	10.9	11.8	12.7	13.6	14.7	15.9	......
92	34	1.41	10.6	11.7	12.5	13.3	......	......	......	......
93	34	1.41	10.8	11.5	12.2	12.9	......	......	......	......
94	35	1.41	10.0	10.8	11.4	12.1	......	......	......	......
105	35	1.35	10.9	12.0	13.0	14.0	......	......	......	......
106	35	1.35	9.5	10.2	10.8	11.3	......	......	......	......
Average	38	1.44	10.3	11.2	11.9	12.7	13.6	14.7	15.9	......
GROUP 6.										
89	35	2.30	10.9	11.4	11.7	11.9	......	......	......	......
95	35	2.04	11.3	12.0	12.8	13.8	......	......	......	......
96	35	2.02	10.5	11.5	12.4	13.1	......	......	......	......
Average	35	2.12	10.9	11.6	12.3	12.9	......	......	......	......

uniformity than those of s_2 and e, and a few have been thrown out of the general averages where they appeared to differ widely and erratically from the others of the group. In Table 7 the columns are grouped as before.

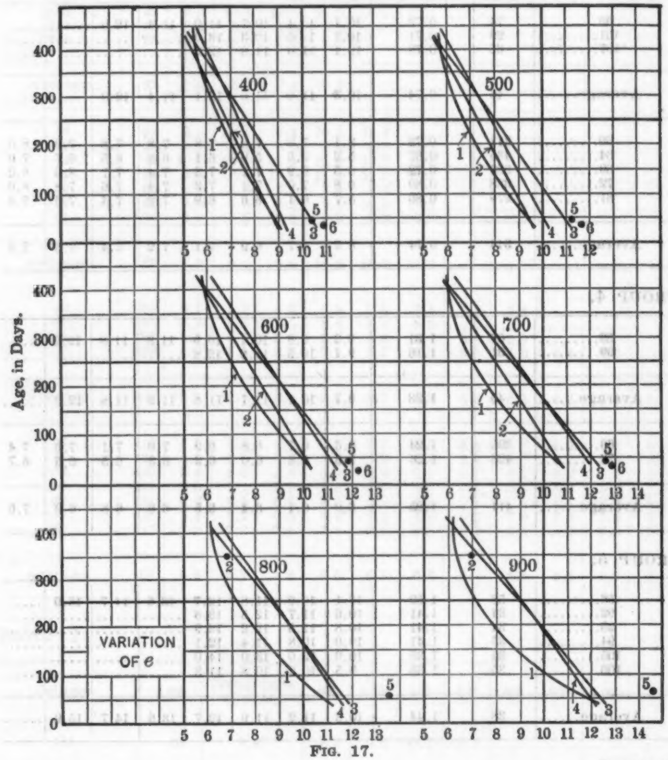

FIG. 17.

The average values from Table 7 are plotted on Fig. 18; with ages, in days, as ordinates, and values of m as abscissas. The single column of Group 2, tested at short time, gives values which for the lower stresses appear to be too high, but the other points lie fairly close together, the value 7 for long-time and 8 for short-time representing fair averages. On the 800 and 900-lb. curves the value for long-time drops below 7. In the few cases where this constant has been used in

discussions, as noted by the writers, it has been assumed as equal to 4, the value derived by Grashof under certain assumptions made by Saint Venant. Estimates based on this would evidently be considerably in error. The value, 7-8, as here found is not surprising when compared with the value for steel, which is about 4.

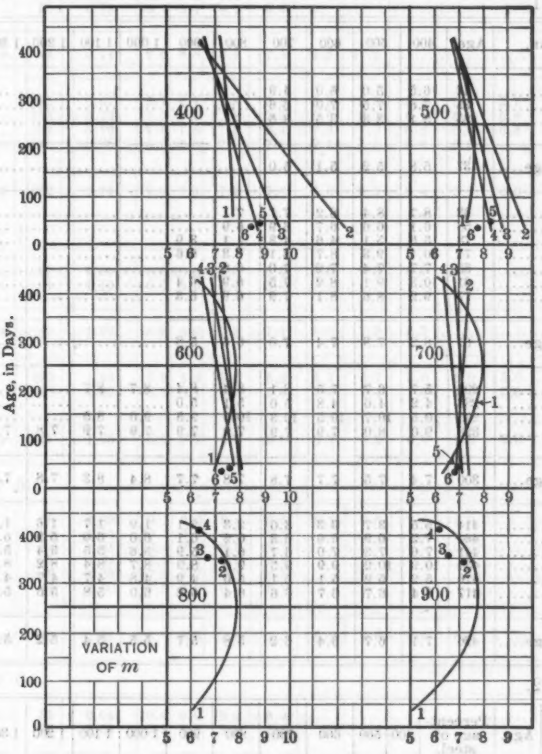

FIG. 18.

As compared with steel, concrete is lacking in density, has numerous small voids, and is granular in nature, with possibly a state of tension existing in the interstitial masses of cement due to the shrinkage of the latter in setting, under certain conditions. The first effect of applying a compressive stress to it would probably be to decrease

the volume by linear deformation in the direction of the stress, with little or no lateral enlargement. If this should actually take place, one would expect the value of *m* to be infinite at the beginning of application of load, and to decrease as the load increased, giving a

TABLE 7.—VALUES OF *m*.

GROUP 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.......	42	6.5	5.0	5.0	4.9	......	......	......	......	......	......	......
114.......	35	7.6	7.5	7.0	6.6	......	......	......	......	......	......	......
115.......	35	3.1	3.3	3.5	3.5	......	......	......	......	......	......	......
Average....	37	5.8	5.2	5.1	5.0	......	......	......	......	......	......	......
38.......	68	8.7	8.4	8.2	7.7	7.1	......	......	......	......	......	......
42	58	6.1	6.6	6.7	6.9	6.9	......	......	......	......	......	......
43.......	73	5.5	5.1	4.6	4.3	4.1	3.9	......	......	......	......	......
44.......	74	10.1	9.3	8.7	8.1	7.3	6.6	......	......	......	......	......
50.......	65	7.7	7.4	7.2	7.0	6.8	......	......	......	......	......	......
58.......	61	10.3	9.1	8.2	7.5	6.9	6.4	......	......	......	......	......
61.......	77	9.2	8.6	8.1	7.9	6.9	6.3	......	......	......	......	......
Average....	67	8.2	7.8	7.4	7.0	6.6	5.8	......	......	......	......	......
46.......	296	5.7	6.7	7.5	8.1	8.4	8.4	8.7	8.7	......	......	......
47.......	288	4.2	4.6	4.8	5.0	5.1	5.0	......	......	......	......	......
53.......	295	10.5	10.7	10.5	10.3	10.0	9.5	9.0	8.5	......	......	......
67.......	323	9.0	8.0	7.9	7.9	7.9	7.9	7.9	7.9	7.8	7.5	7.2
Average....	300	7.4	7.5	7.7	7.8	7.8	7.7	8.4	8.3	7.8	7.5	7.2
48.......	414	5.5	3.7	8.3	8.0	2.3	2.1	1.9	1.7	1.6	1.5	1.5
51.......	434	7.2	6.9	6.6	6.3	6.2	6.1	6.0	6.0	5.9	5.8	5.7
54.......	441	7.6	7.3	7.0	6.7	6.1	5.9	5.6	5.5	5.4	5.3	5.2
62.......	434	10.9	10.2	9.9	9.5	9.0	8.9	8.7	8.4	8.2	8.0	7.8
79.......	416	5.2	5.2	5.1	5.1	5.0	4.9	4.8	4.7	4.7	4.6	4.5
80.......	417	6.4	6.7	6.7	6.6	6.4	6.2	6.0	5.8	5.5	5.3	5.0
Average....	426	7.1	6.7	6.4	6.2	5.8	5.7	5.5	5.4	5.2	5.1	5.0

GROUP 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
111....	35	0.47	12.5	9.5	7.8	6.9	......	......	......	......	......	......	......
71....	317	0.45	8.7	8.7	8.6	8.4	8.1	8.0	8.0	8.0	8.0	7.9	7.8
75....	300	0.54	8.3	8.2	8.1	8.0	7.9	7.8	7.7	7.5	7.3	7.0	6.8
82....	413	0.52	5.2	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.3	5.3	5.3
Average.	343	0.50	7.4	7.4	7.4	7.3	7.2	7.1	7.0	7.0	6.9	6.7	6.6

TABLE 7.—(*Continued.*)

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
GROUP 3.													
66....	74	0.72	14.4	13.3	12.6	12.0	11.6	11.2	......	......	......	......	......
101....	29	0.71	8.7	7.8	7.1	6.2	......	......	......	......	......	......	......
107....	85	0.78	5.7	5.1	4.7	4.2	......	......	......	......	......	......	......
Average.	46	0.74	9.6	8.8	8.1	7.5	11.6	11.2	......	......	......	......	......
64....	485	0.67	5.7	5.7	5.8	6.0	5.9	5.8	5.8	5.8	5.7	5.6	5.5
69....	323	0.72	7.9	8.0	7.8	7.2	6.6	6.3	6.1	6.0	5.8	5.7	5.5
72....	383	0.72	7.8	7.8	7.6	7.4	7.2	7.0	6.8	6.6	6.4	6.1	......
81....	270	0.86	6.6	7.0	7.7	7.2	7.1	7.0	6.8	6.5	......	......	......
Average.	352	0.74	7.0	7.1	7.2	7.0	6.6	6.5	6.4	6.2	6.0	5.9	5.5
GROUP 4.													
59....	67	1.31	11.6	10.8	9.9	8.9	8.0	7.3	6.7	......	......	......	......
99....	31	1.16	6.0	5.9	5.7	5.3	......	......	......	......	......	......	......
Average.	49	1.23	8.8	8.3	7.8	7.1	8.0	7.3	6.7	......	......	......	......
63....	395	1.24	5.3	5.3	5.2	5.2	5.1	5.1	5.0	5.0	5.0	......	......
73....	428	1.26	8.1	8.1	8.0	7.9	7.7	7.4	7.1	7.0	7.0	6.9	6.9
Average.	411	1.25	6.7	6.7	6.6	6.5	6.4	6.2	6.0	6.0	6.0	6.9	6.9
GROUP 5.													
56....	58	1.69	9.1	9.0	8.8	8.5	8.1	7.6	7.0	6.3	......	......	......
92....	34	1.41	10.1	9.0	8.0	7.2	......	......	......	......	......	......	......
93....	34	1.41	10.4	9.3	8.3	7.3	......	......	......	......	......	......	......
94....	35	1.41	8.2	7.7	7.2	6.6	......	......	......	......	......	......	......
105....	49	1.35	7.9	7.7	7.1	6.6	......	......	......	......	......	......	......
106....	44	1.50	7.5	7.0	6.4	5.9	......	......	......	......	......	......	......
Average.	42	1.46	8.9	8.3	7.6	7.0	8.1	7.6	7.0	6.3	......	......	......
GROUP 6.													
95....	35	2.04	10.4	9.6	9.0	8.4	......	......	......	......	......	......	......
96....	35	2.02	9.9	8.5	8.0	8.0	......	......	......	......	......	......	......
Average.	35	2.03	10.2	9.1	8.5	8.2	......	......	......	......	......	......	......

curve such as is found in Columns 52 or 62. Generally, however, the curve runs from a lower value up to a maximum at a stress which roughly averages 275 lb. per sq. in., but which may be anywhere between 100 and 500 lb., or may not be clearly defined at all. The

points up to the 500-lb. stress are irregular in location, and it is often impracticable to lay a fair curve through them, indicating that the material is settling together somewhat irregularly and adjusting itself under the imposed load, not uniformly, as would a homogeneous mass, but somewhat spasmodically, until a stage is reached where the mass is thoroughly compacted, and all the initial stresses are relieved; from this point the deformation becomes quite consistent, as is shown by the values of m becoming quite regular, the points varying but slightly from a fair curve.

This same tendency to irregularity is shown slightly in the s_2 curve, but far more plainly in the e curve, where the points are irregularly placed, frequently showing a considerable break in continuity at some point. This irregularity of e seems to be without any law, except that it disappears at stresses of from 200 to 400 lb. There is no reason to suppose that slipping of the extensometer would be more likely to occur here than at higher stresses, and the fact that the values of e, which depend only on the s_2 readings, show an irregularity similar to that of m would indicate that this is not due to slip in the instrument.

It is the conclusion of the writers, therefore, that below about 400 lb. per sq. in., the concrete deforms under the load quite irregularly and without any law; that at about 400 lb. the causes of this irregularity are largely eliminated, and that thereafter the action conforms closely to a definite law which can be represented by a fair curve until some higher limit is reached; which limit lies well beyond ordinary working stresses, and is not defined by these tests. At this limit the destruction of the cement bond would begin, and the curves would become irregular again.

Conclusions.

From the foregoing results the writers deduce the following conclusions, as applicable to the materials used in these tests:

(1) The value of Poisson's ratio is approximately 8 for short-time and 7 for long-time tests.

(2) The value of e is in general below 10, only exceeding that value for short time and high stress.

(3) Spiral reinforcement does not, by its restraining effect on the core, perceptibly affect the values of *m* or *e* for stresses below 900 lb. per sq. in.

(4) Spiral reinforcement does not perceptibly assist the concrete in carrying the load within the limits of ordinary working stresses.

(5) The limit at which the first visible sign of failure occurs is raised by spiral reinforcement. This effect is greatest in short-time tests, and is perceptible for all percentages of reinforcement.

(6) The plain column fails abruptly, giving no warning of approaching collapse. The hooped column may be depended on to give warning of approaching failure at from 70 to 90% of the ultimate strength, when not less than 0.50% of steel is used.

(7) The deformation is quite irregular up to about 400 lb. per sq. in.; above this it is quite regular up to a limit which is not defined by these tests, but is well above allowable working stresses.

The tests indicate, but cannot be said to prove conclusively, that:

(8) The ultimate strength of the column is increased by spiral reinforcement by an amount roughly proportional to the quantity of steel used, up to the limit of 1.58%; and above this limit the ultimate strength is simply the amount which can be carried by the granular core supported by the spiral, but with no cohesion of its own.

DISCUSSION

Mr. Godfrey.

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—This is a valuable series of tests. It is unfortunate that such information, and proper interpretation and digestion of the same, were not available before common practice in reinforced concrete design was crystallized into the deplorable shape that it has taken in all building codes and in the report of the Joint Committee on Concrete and Reinforced Concrete, as well as in the standard books on the subject.

"Some experiments then made on the ultimate strength of columns with longitudinal steel bars embedded in the concrete indicated that such reinforcement, unless hooped or banded at frequent intervals, could not be relied on to increase the strength of the column, and in some cases might even render its strength less."

It is gratifying to know that truths, the recognition of which the writer has been seeking since 1906, are beginning to dawn on the Profession. He predicted that it would take 10 years. It will probably take that before the Joint Committee and the authors of building codes recognize the truth. Books will follow some years later, and the rodded column will have eked out its murderous existence.

The writer has followed this subject pretty closely, and, so far as he is able to learn, no one but himself had condemned the rodded column until recently, when it was criticized severely at a meeting of an engineering society in Chicago. It is absolutely true, as the authors state, that years ago experiments indicated that longitudinal rods do not reinforce a column, but it is just as true that the very authorities who made these experiments and who discussed them in their books, endeavored to explain away the facts and to apologize for the particular specimens of concrete in the weakened so-called reinforced columns. These same authorities recommend the addition of from 15 to 39% of strength for every 1% of this weakening steel.

What are the facts to-day? Every building code allows rodded columns; practically all the standard books recommend them; the Joint Committee Report allows them; nearly every great wreck has had them, and has been the result of them.

The authors are wise to ignore rodded columns in their tests and investigation, for the rodded column has absolutely nothing to recommend it to any intelligent man, and there can be no rational incentive to make tests on them except for the purpose of demonstrating their unfitness for structures, the unfitness that an intelligent analysis of the column will demonstrate, entirely apart from tests. It is deplorable that, though the Profession has been in possession of this experimental demonstration for a number of years, the facts have been so

* Pittsburgh, Pa.

perverted, and recommendations have been so utterly contrary to what they have shown, that standard methods of constructing columns are criminally bad. Mr. Godfrey.

The writer would put quite a different interpretation on the tests made by the authors from that given in their paper. Their comparison between the plain and the hooped column is unfair to the former. Not that the plain column has any right to consideration as a structural member, but such comparisons give the hooped column an apparent advantage over other structural members, in the matter of unit stress, which it does not possess.

In calculating the unit load at first sign of failure, they have taken only the area of the core for columns with hooping, whereas they take the full sectional area of the plain columns. The area of the core is about 10 sq. in. and that of the full section is about 13 sq. in. Making this approximate correction on all their columns having embedded coils, the writer has discovered the following remarkable facts: The average load at first sign of failure for the 24 plain columns is 2 250 lb. per sq. in.; the average load at first sign of failure for the 18 columns having less than 1% of hooping is 2 250 lb. per sq. in.; the average load at first sign of failure for the 28 columns having more than 1% of hooping is 2 230 lb. per sq. in.; and the corresponding average values for the ten highest in the foregoing groups are:

Plain columns, 3 057 lb. per sq. in.
Less than 1% hooping, 2 614 lb. per sq. in.
More than 1% hooping, 2 769 lb. per sq. in.

If these facts teach anything at all, they blazon out the truth that hooping has no influence whatever on the first failure of a column, and it is the first failure that is really the ultimate failure. A column is useless after it begins to spall on the surface.

The data presented by the authors on the ultimate strength of these columns are only of academic interest. They are totally erratic, and simply represent the post-mortem strength of columns which have already failed. How long it may take a rattlesnake's tail to cease wagging after its head is crushed, has no bearing on the safety of the persons who were menaced by the live snake. The load-resisting power of a more or less disintegrated mass of concrete encircled by a steel spiral has no bearing on the design of a safe structure, if the first sign of failure is ignored. The authors might as well take one series of columns and label them "A" and use their full area in determining the unit load, and then take another set which have gone through some treatment or other and label them "B", and use only 80% of their area, and then say, when Columns "B" stood exactly the same load as Columns "A", that Columns "B" were 25% better than Columns "A", as to make the comparison that Table 3 shows on its face.

Mr. Godfrey.

We have it on the authority of Ernest McCullough, M. Am. Soc. C. E., that in a large number of commercial designs he has never found one which excluded the outer shell in the strength of the column. The writer has persistently refused to have anything whatever to do with the checking of rodded column installations, or doubtless he would have discovered the same thing. There is no reason why investigators should omit the outer shell of a column, either in making comparisons or in discovering the strength of columns.

These tests demonstrate several things. One is that the closer the pitch of the spirals, in general, the greater the ultimate strength of the column. This could be reasoned out by analogy. A brick will stand more load per square inch flatwise than on end. A mortar joint is stronger the thinner it is made. A flat disk will stand much more load than a high cylinder of the same diameter. There is a limit, however, beyond which this added strength is of no use. It would be unsafe to have a column in a structure with the shell outside of the coil spalled off. Hence the "first sign of failure" is the function that must govern in safe design. This value, in these tests, runs from 1 400 lb. per sq. in. up, on the full area in compression, on the hooped columns, or 1 800 lb. on the core.

Now what factor of safety will the Pittsburgh Building Code give, for example, allowing as it does from 1 300 to 1 750 lb. per sq. in. of alleged safe load on columns, as shown by T. L. Condron, M. Am. Soc. C. E.* Other building codes are not quite as bad as that of Pittsburgh, but those of St. Louis and Minneapolis are close seconds. The gravel concrete common in Pittsburgh is not nearly so strong in cylinders as the limestone concrete used by the authors.

In a testing machine, a column might show a certain strength at first failure, and two or three times as much at ultimate failure, but, if it begins to fail in a building, the sway and eccentricities unavoidable in structures will doubtless mean its ultimate failure.

One of the purposes of a factor of safety is to cover imperfections. What would happen in this "safe" Pittsburgh column, if a workman should drop a wooden block into it accidentally, or if there should be a void, filled up later by a trowel full of mortar? Who would be responsible for the possible general wreck of the building, the workman, the designer, or the commercial interests who get up these building codes? Laboratory tests are more carefully made than actual construction, so that this should be taken into account.

These tests demonstrate that there is no definite relation between the percentage of steel reinforcement and even the ultimate strength of the core. This, together with the facts concerning the pitch of the coil, shows clearly, what the writer has for many years contended,

*Paper before the Western Society of Engineers, Feb. 9th, 1914; *Engineering News*, Feb. 19th, 1914; *Engineering Record*, Feb. 21st, 1914; *Engineering and Contracting*, Feb. 18th, 1914.

that Considère's formula is wrong. That formula, and it is used in the newest building code, which is that of Pittsburgh, would add to the strength of a column having a given pitch by adding to the size of the wire in the pitch, a perfectly absurd proposition, as the wire might already be strong enough to stand the crushing strength of the disk between two coils. Mr. Godfrey.

Another important fact brought out by the authors is that the ratio between the moduli of elasticity of steel and concrete is not more than 10. The value, 15, is very commonly used, with a resultant degradation of design, where steel rods are considered in compression.

The authors state:

"That the shrinkage of the concrete, causing initial compression in the steel, might be the cause [of greater deformation in hooped columns] is untenable, because the concrete was immersed in water during the entire aging period."

It is presumed that the columns were dry when tested. Tests show that specimens shrink and swell by the mere process of drying and soaking them.

What the Profession needs is to ascertain the combination of spiral and longitudinal reinforcement for a column of given diameter, which will be consistent and properly balanced. This should then be adopted as a standard, and the value of a column should be based on the kind of concrete used, just as in beams or slabs. The longitudinal steel should be stiff enough and the coils should be close enough to hold these rods from buckling. The writer* advocated this in 1906 and worked out such a standard.

The Profession is not in need of tests so badly as of proper interpretation of those already made. This is not said in disparagement of those made by the authors. They are of great value. What is sorely needed is complete revision of building codes, books, and of the Joint Committee Report. The present standards of reinforced concrete design are dangerous and outrageous.

A. W. Buel,† M. Am. Soc. C. E.—It appears to the speaker that the programme for this series of tests was planned so that results or conclusions of much practical value or reliability could hardly be expected, at least none commensurate with the expenditure. Mr. Buel.

The slenderness ratio, about 6.75 for the concrete or 7.7 for the coil, was much too small to represent conditions in practice, and should classify the series with compression tests rather than with column tests.

The concrete mixture, 1 : 2 : 4, was too rich to use economically with spiral or hoop reinforcement, and the tests do not show the true value of such reinforcement, which is greater with lean concretes.

* In his book entitled "Concrete."
† New York City.

Mr. Buel. Some indication of this is shown by a comparison of the short- and long-time tests of this series.

The section of the test columns was so small that the results will hardly be depended on as a basis for structural designing, in view of the fact that engineers already have reports of a considerable number of full-sized tests. Some 6 or 8 years ago, when very few reports of full-sized tests were available, such tests as these would have been received as a valuable contribution. A dozen full-sized column tests, carried out on a well-planned programme, would be a more valuable addition to the experimental data on the subject than several hundred tests on small model specimens having dimensions entirely out of proportion to those generally found in practice.

The pitch of the spirals varied from 0.25 to 0.75 in., in columns of the same group. This, and also the variations in age, make it impossible to compare fairly the results from one test column with those from another, to draw reliable conclusions, or, for practical purposes, to realize what the authors describe as the "Aim of the Tests".

The records of "first sign of failure", particularly for the short-time tests, do not agree with tests reported by other experimenters, where the envelope of concrete or cement outside of the spiral or hooping almost invariably began to show signs of failure as soon as the load reached or exceeded the ultimate load of a similar column without reinforcement. In other words, all previous tests have indicated that spiral or hoop reinforcement does not materially raise the point of first sign of failure. Mr. Godfrey has pointed out what seems to be an error in computing the intensity of stress at first sign of failure: that the area of cross-section enclosed by the spiral was used, whereas the area of the entire column, including the concrete outside of the spiral, should have been taken for this computation. For the determination of the intensity at the ultimate load, the area enclosed by the spiral is correct, because at that time the outside envelope would have been destroyed.

In a number of cases the pitch of the spirals was somewhat higher than that recommended by Considère for maximum efficiency of hooping. This was confined mostly to columns with the smaller percentages of spiral reinforcement.

Most of the eight conclusions drawn by the authors from the results of this series of tests are in substantial agreement with those deduced from previous experiments, and can be considered as a confirmation of the latter.

Conclusion No. 2 gives a value for e, the ratio of the modulus of elasticity of the steel, E_s, to that of the concrete, E_c, which is remarkably low, considering what engineers have been accustomed to use and the results of previous experiments. The speaker would like to ask the authors whether the value of E_c, which they derive from

the formula, $600\,000\,f_c \div s_2$, is the true value of E for concrete alone, and is it not possible that it was affected by the spiral reinforcement to some extent? Also, what was the value of E_c for the plain concrete columns without reinforcement? Is not the E_c computed by the authors' formula, the E for the column as a whole, rather than the E for the plain concrete? Mr. Buel.

Conclusion No. 5, that the spiral reinforcement materially raises the limit of the first sign of failure, will require further experimental demonstration to be fully convincing, in view of the considerable number of determinations which have shown different results.

C. G. Wrentmore,* M. Am. Soc. C. E. (by letter).—The denunciation by Mr. Godfrey of the column reinforced with longitudinal rods is more emphatic than is considered to be justified by either our experience or by the results of analysis, so far as the latter can be applied. These tests were begun more than 7½ years ago, and prior to that time the writer had concluded that for such columns the treatment then current was unsound, and that he would make no further use of them except with thorough hooping. Mr. Wrentmore.

There are cases, however, where the use of longitudinal rods in members designed primarily for compression is justified. The writer's experience in the Philippines, where high wind pressures and mild earthquake shocks are frequent, and where violent earthquakes may be anticipated, at least in some localities, has demonstrated that reinforced concrete is peculiarly well fitted to withstand the resultant stresses. Although he would use longitudinal rods to resist the bending stresses, they would not be included in designing for direct compression, except where hooped at frequent intervals.

Neither is the writer inclined to concur in the logical conclusion derivable from Mr. Godfrey's statement that "though the Profession has been in possession of this experimental demonstration for a number of years, the facts have been so perverted, and recommendations have been so utterly contrary to what they have shown, that standard methods of constructing columns are criminally bad". It is undoubtedly true that in the past, much work has been done, in reinforced concrete, which is unsound, and that in the future, faulty designs may be drawn; but the possibilities and limitations of reinforced concrete are not to be learned in a day, and the Profession is rather to be congratulated that the history of its use shows so few failures, than to be accused of a lack of either acumen or honesty in the interpretation of experimental data.

As to Conclusion (5) of the paper, that "The limit at which the first visible sign of failure occurs is raised by spiral reinforcement. This effect is greatest in short-time tests, and is perceptible for all percentages of reinforcement": Mr. Godfrey says (after reducing

* Manila, Philippine Islands.

Mr. Wrentmore.

certain loads to unit stresses on the full section of the hooped column, instead of on the core) "if these facts teach anything at all, they blazon out the truth that hooping has no influence whatever on the first failure of a column * * *".

The writer holds that the tests, as given, demonstrate: (*a*) If the sectional area of the hooped column is taken as the effective area, Conclusion (5) is plainly true; (*b*) If the sectional area of the hooped column is taken to include the concrete outside the hooping, the long-time tests show no increase due to hooping, but the short-time tests show an increased strength which is greater with the larger quantities of steel. In Table 8, the writer compares the unit stress at first sign of failure of the columns of the various groups, basing it on the full section of all columns.

TABLE 8.—UNIT STRESS AT FIRST SIGN OF FAILURE.

Group	1	2		3		4		5		6	
Percentage of reinforcement	0.0	0.44–0.54		0.67–0.86		0.16–1.35		1.41–1.69		2.02–2.32	
Age.	Stress.	Age.	Stress.	Age.	Stress.	Age.	Stress.	Age.	Stress.	Age.	Stress.
40	1 089	35	1 650	46	1 578	45	1 970	51	1 980	42	2 020
68	1 796	......	......	......	......	......	......	......	......	......	......
318	2 763	336	2 495	345	2 490	365	2 781	277	2 980	393	2 100
428	2 908	......	......	......	......	......	......	......	......	......	......

Table 8 shows that not only for an age averaging roughly 45 days is the strength increased by hooping, but also that this increase is very considerable, the smallest amount shown being that of Group 3, where it is 45% for from 0.67 to 0.86% of steel. (The 1 650 of Group 2 is given by a single column, and is abnormally high.)

For this age the lowest value for a single column is 1 650 for Group 2; 1 255 for Group 3; 1 330 for Group 4; 1 915 for Group 5; and 1 680 for Group 6; and the minimum for Group 1, plain columns, is 815; so that, if minimum values are taken, the increase is 54% with from 0.67 to 0.86% of steel. Moreover, Mr. Godfrey says:

"Hence the 'first sign of failure' is the function that must govern in safe design. This value, in these tests, runs from 1 400 lb. per sq. in. up, on the full area in compression, on the hooped columns, or 1 800 lb. on the core."

The hooping will be more effective in raising the first sign of failure as the modulus of elasticity decreases, or, more exactly, as the deformation under a given unit load increases. This would result in a greater effect in a weak mixture and also in green concrete,

Mr. Wrentmore.

and less in a strong mixture, and would gradually decrease with age. As the tests show that the increase is about 45% at 45 days, it should be expected that for 20 or 30 days, it would be even more.

The fact that economy calls for stripping the forms at an early date, that under actual working conditions they are frequently removed as early as 14 days, thus throwing the load on the concrete while the elastic modulus is still quite low, makes this increase of the limit of first sign of failure especially useful.

The weakness of the concrete at this time is due to the weakness of the cement, and supplementing the bonding action of the cement by the use of a spiral of steel, the strength of which is accurately determinable while that of the green concrete is always questionable, gives the structure an insurance against failure at the time when failures have usually occurred. The large deformation which may occur on stripping forms from a concrete which is slow in hardening, throws the spiral into stress, and so puts it in condition to exercise a greater influence at any later date. Again, the percentage of ultimate strength at which the first sign of failure occurs is lower at the earlier age, and this fact acts as an insurance to workmen, plant, and building against an abrupt collapse without any warning. Such collapse involving an entire structure, may result from weakness in, or overloading, a single column; whereas, if the member, by flaking or spalling, had given warning that failure was impending, much damage might have been averted. It is under precisely these conditions that the column with longitudinal rods is especially undesirable, as the large deformation of green concrete induces high compressive stresses in the steel which tend to cause buckling, so that the steel may actually weaken rather than strengthen the column, unless the rods are hooped at frequent intervals.

The relation of ultimate strength to first sign of failure is of interest only in so far as it may prove to be an insurance against loss of life or property by giving warning of imminent failure. No engineer can be sure that his structure will not at some time, through ignorance or carelessness, be subjected to loads in excess of those for which it was designed, and it is not undesirable that in such case the structure be capable of giving warning of its overstressed condition rather than to fail abruptly.

Mr. Buel's inquiry regarding the method of determining the value of E is answered fully in the paper. An examination of Fig. 17 will suffice.

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Paper No. 1315

THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE.*

BY CHARLES H. LEE, ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. JAMES OWEN, G. E. P. SMITH, O. E. MEINZER, KENNETH ALLEN, ROBERT E. HORTON, AND CHARLES H. LEE.

SYNOPSIS AND CONCLUSIONS.

The objects of this paper are to show the possibility and practicability of measuring the annual rate of recharge of underground reservoirs of the closed-basin type, and to indicate broadly the factors which determine safe yield from a basin by artificial development, such as Artesian flow or pumping.

The paper opens by pointing out the importance of the problem in California and the Southwest. Following this there is a description of the physical features of underground reservoirs and the general principles of inflow, outflow, and storage. The body of the paper presents detailed methods and results of extended measurements, by the Los Angeles Aqueduct Bureau and the United States Geological Survey, for the determination of the rate of annual recharge of the Independence Basin, in Owens Valley, California. The subjects of percolation from stream channels, relation of precipitation and altitude, soil evaporation combined with transpiration from grass, and ground-water fluctuations, were carefully studied in the field, and original data are pre-

* Presented at the meeting of May 6th, 1914.

sented. The paper closes with a discussion of the relation which the net safe yield from a basin bears to the rate of annual recharge.

The conclusions are as follows:

1.—The "underground reservoirs" of California and the Southwest are water-tight rock basins, represented by the topographic valleys, which are filled with porous alluvial material in which the voids are saturated with water.

2.—Inflow into these basins is by percolation from water on the surface of the alluvial filling, which source may occur as direct precipitation, stream flow, irrigation, or flooding. Natural ground-water loss occurs in the region of lowest depression of a basin, and consists of the breaking out of water at the surface in springs or seepages, evaporation from soil, transpiration, and underflow. Artificial development, by wells or other methods, reduces the natural ground-water loss. Considered as averages, the rates of recharge and ground-water loss are equal, unless the artificial draft is excessive.

3.—The rate of recharge in a region of small precipitation and high evaporation rate can be determined most accurately, and with least expenditure of time and money, by measuring the elements which make up the ground-water loss. Of the natural elements, the most important are soil evaporation and transpiration. The underflow is relatively small and often negligible.

4.—The safe yield of artificially developed ground-water obtainable from an underground reservoir is less than indicated by the rate of recharge, the quantity depending on the extent to which soil evaporation and transpiration can be eliminated from the region of ground-water outlet.

Introduction.

There are in California and the arid States of the Southwest many valleys underlaid by porous alluvial material in which the voids are filled with water. The ease with which water can be developed from wells in these valleys and the definite bounds of the water-bearing formation have led to the use of such terms as "underground lake," "underground basin", or "underground reservoir". These terms are in general use among local hydraulic engineers, and have been adopted by the California Courts in numerous recent decisions pertaining to the use of diffused percolating water occurring in closed basins.

A problem which is being presented to the Engineering Profession for solution is the determination of the safe yield of "underground reservoirs", or the net annual supply which may be developed by pumping and Artesian flow without persistent lowering of the ground-water plane. The answer to this problem must soon be had throughout the Southwest, and particularly in Southern California, where the use of underground water has advanced most rapidly. The available surface supplies of the region are now used so extensively that future extension of irrigation must depend on the underground supply. Already, however, the growing popularity of ground-water supply for irrigation and the heavy drafts made possible by improved pumping machinery and cheap power are giving rise to conditions of dangerous overdraft on many of the so-called inexhaustible underground water supplies. Furthermore, in many of the sparsely settled valleys of the Southwest, where very limited ground-water supplies are available, preparations are being made to develop pumped water for irrigation far in excess of the safe yield. The writer has in mind such a valley, where, out of 90 000 acres of agricultural land, filed on in good faith under the provisions of the Desert Land and Homestead Acts, it can be said with reasonable certainty that not more than 2% can ever be put under cultivation. In addition to the use of underground water for irrigation, it is being developed extensively for municipal purposes. The City of Los Angeles derives its present supply entirely from an underground reservoir, the San Fernando Valley, and is preparing to develop a similar supply in Owens Valley to be held as a reserve in connection with the Los Angeles Aqueduct. A portion of the supply of both Oakland and San Francisco is developed from underground sources, and the possibility of increasing largely the ground-water supply derived from Livermore Valley for the latter city has been the subject of considerable debate among prominent members of the Society. The problem, therefore, is an important one, and on it depends, not only the safe investment of capital, but also the very life of large industries and communities.

The sources of underground water are so difficult of measurement and its movements are so hidden from view, that the solution of the problem, until very recently, has been merely a subject for speculation and theory. Within the past few years, however, the study of underground water supply has been given considerable attention by the

United States Geological Survey as well as by engineers who have had these problems to meet. Although the fact of the existence of these "underground reservoirs" has been established, their sources of supply and outlets recognized, and many data regarding well fluctuations have been accumulated, yet very little has been done toward developing methods of measuring the rate of recharge or studying the factors which limit the quantity of water which can be safely developed from underground reservoirs.

The writer has had opportunity to investigate a number of the important underground reservoirs of California, and in this paper he presents certain general principles which seem to him to be justified by the existing data. Although these principles may seem to be self-evident, yet the writer has no knowledge that they have ever been applied to the practical solution of the problem in hand. To show the possibilities of their application, therefore, he presents data and studies for an underground reservoir in Owens Valley, California, where it was desired to ascertain the quantity of ground-water that could be developed safely without overdraft. Much of this information has already appeared in print* in greater detail, but the writer believes that the subject is of sufficient importance, and the component studies are of wide enough technical interest to be presented to the members of the Society for discussion and expression of opinion.

General Principles.

The typical underground reservoir is, geologically, a structural basin filled with alluvial débris from the adjoining mountain ranges. These basins are the product of faulting accompanied by the uptilting of a crustal block from one side of the line of fracture. The formation is very common throughout the Southwest, reaching its most perfect development in the Great Basin region of Utah and Nevada, where the name "Basin-Range" has been applied to it. In California the basins are found in the valleys of the Coast Range and along the base of the Sierra Nevada, Sierra Madre, San Bernardino, and San Jacinto Ranges. The rock enclosing these basins is in most cases impervious to water and practically insoluble. Along the coast of California, shales and cemented gravels predominate, and are practically non-water-bearing in comparison with the porous gravels

* Water Supply Paper No. 294, U. S. Geological Survey, 1912.

filling the basins; and, in the interior of the State, the enclosing rock formation is largely granite. Most of the basins can be considered as closed except for a subterranean outlet usually known as the "Narrows". This occurs at the lowest point in the rock rim, where the gravels contract into a neck filling a narrow depression or canyon cut into the confining rock. The quantity of underground water escaping through such an outlet is usually very small, however, as has been shown by a number of well-known underflow observations. Hence, the underground reservoirs can generally be considered as closed rock basins, the effective storage capacity of which is the void spaces between the particles of sand and gravel with which they are filled.

The usual sources of supply for underground reservoirs are percolation from flowing surface streams, from precipitation, or, where the supply is not ground-water derived from the basin, from irrigation on the surface of the porous gravels. The water thus absorbed sinks downward to the general ground-water plane and then moves laterally toward the region of lowest depression. This region, in contrast to the surrounding dry soil or desert, is usually characterized by springy, swampy conditions, and is commonly known in Southern California as a cienaga. The natural outlets for underground water are by springs or seepages discharging into the surface channels which drain the cienaga, by evaporation from damp soils and vegetation within the cienaga, and, to a limited extent, by underflow from the basin. The surface streams formed by the oozing out of underground water join to form a larger stream, which in all respects corresponds to the outlet of a lake or reservoir, and, passing from the basin, pursues its course just as any other surface stream. Its flow is characterized by permanence and regularity, except as it is augmented by surplus flood water which, during a limited period following winter storms, passes from the basin without being absorbed by the gravels.

The general principles of inflow into and outflow from an underground reservoir of the type described correspond with those of surface reservoirs. The difference lies in the relative speeds with which the general water surface assumes a horizontal position following increase or decrease of volume stored. In the case of a surface reservoir or lake, the effect of inflow or outflow is an immediate complete readjustment of surface level. The frictional resistance offered by the

particles filling an underground reservoir is so great, however, that the movement of water from an area of high level is very slow, varying from a few hundred feet to a few feet per day, depending on local conditions. As a result, the water surface in an underground reservoir is never horizontal, being steepest near the mouths of the mountain canyons, the run-off from which is the most important source of supply; it is most nearly horizontal at the region of outlet; and varies in slope and elevation from time to time, depending on the rate of recharge.

The average rates of inflow and outflow of an underground reservoir must be equal, otherwise there would be persistent rise or fall of ground-water levels until such a balance is reached. There are, therefore, two possible methods of measuring the rate of recharge. either by determining the total percolation from various sources into the porous material of the basin, or by determining the ground-water losses. The first method is to be preferred where the source of percolation is almost entirely stream flow from which channel losses can be accurately measured; or where the precipitation is large, well distributed through the year, and forms the principal source of supply. The first of these conditions could occur only in an arid region, and the second is typical of humid regions.

The method by determination of ground-water losses is one peculiarly adapted to arid or semi-arid conditions with high evaporation rate, such as exist throughout the Southwest. It has been the writer's observation in this region that soil evaporation and transpiration constitute from 50 to 100% of the ground-water losses from underground reservoirs, the average exceeding 75 per cent. Other losses are largely the flow from springs and seepages, which can be measured with precision. Rates of soil evaporation and transpiration from grasses do not present insurmountable difficulties of measurement under arid conditions. In fact, it has been the writer's experience that satisfactory results with specially designed equipment could be obtained from observations extending over 2 years, although a period of 3 years is preferable. Furthermore, the area from which evaporation occurs and the depth to ground-water at various points within it are not subject to wide fluctuations, and are easily measured. The determination of the rate of recharge of underground reservoirs of the basin type, therefore, is a problem of soil evaporation, transpiration, and stream

flow, all of which processes, with the exception of transpiration from trees, are now capable of measurement with relative accuracy at reasonable cost.

The general method pursued in the Owens Valley studies was to ascertain, by extended field measurements of soil evaporation, transpiration, and spring discharge, the average rate of outflow from the basin. All available evidence seemed to indicate that the basin was closed, so that the rate of outflow equalled the rate of inflow or recharge. As a check, therefore, the rate of inflow into this basin from precipitation, stream flow, and irrigation was also determined. The data are presented under the following headings: Physical Features, Precipitation, Stream Flow, Evaporation and Transpiration, Groundwater, and Rate of Recharge by Percolation.

Physical Features.

General.—The Owens Valley lies in east-central California, along the western border of the Great Basin, and at the base of the steep slope of the Sierra Nevada Mountains, as shown by Fig. 1. Including a northern extension, known as Long Valley, its length is 120 miles, and its width, from crest to crest of confining mountain ranges, varies from 15 to 40 miles. The total area of the valley and its tributary mountain drainage is about 3 300 sq. miles, of which 1 200 sq. miles are desert mountains from which the run-off is negligible, 536 sq. miles comprise the Sierra Nevada slope, which yields a large run-off, and 1 580 sq. miles are the transition slope and valley floor, from which very slight surface run-off occurs. The elevation of the valley floor varies from 8 000 to 3 570 ft. above sea level, the latter being at Owens Lake, the lowest depression of the valley. The average elevation of the crest of the Sierra Nevada is 12 500 ft., with many peaks exceeding this elevation by more than 1 500 ft. The White and Inyo Mountains, a desert range bordering the valley on the east, have an average elevation of 10 000 ft., with peaks reaching 13 000 ft.

The valley is a deep structural trough filled with porous alluvial material derived principally from the Sierra Nevada, and inclosed by impervious rock formations. The steep east face of the Sierra Nevada is the result of faulting accompanied by elevation and westward tilting of a great crusted block. The drainage system of the valley consists of a trunk stream, Owens River, fed by approximately

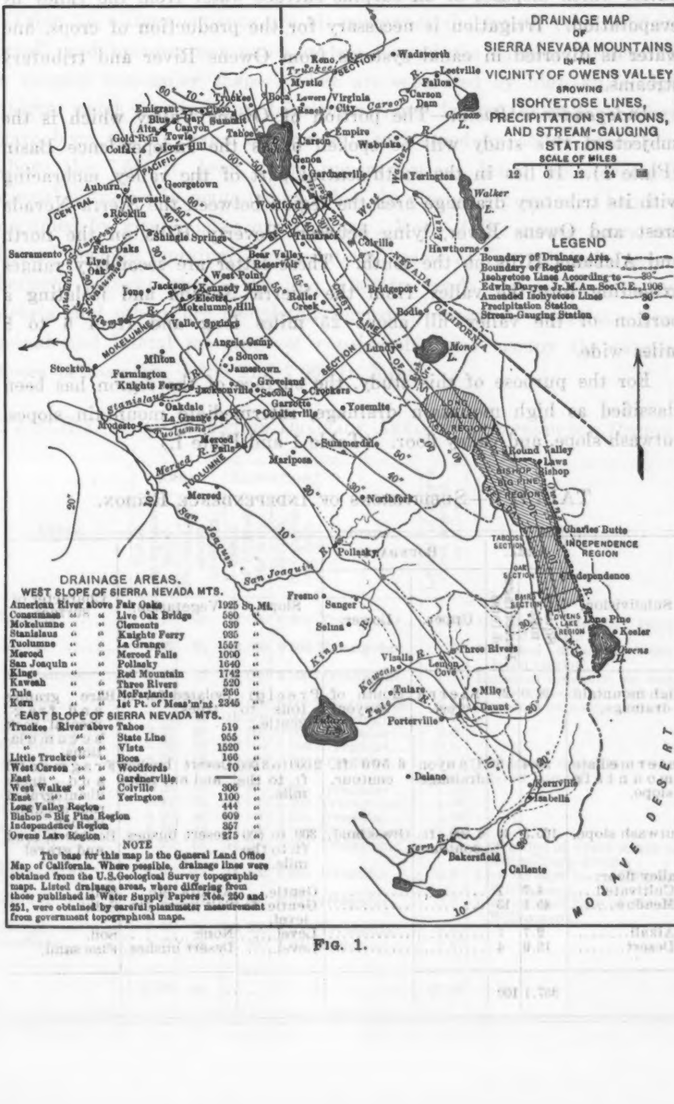

DRAINAGE MAP
OF
SIERRA NEVADA MOUNTAINS
IN THE
VICINITY OF OWENS VALLEY
SHOWING
ISOHYETOSE LINES,
PRECIPITATION STATIONS,
AND STREAM-GAUGING
STATIONS
SCALE OF MILES
12 0 12 24
LEGEND
Boundary of Drainage Area
Boundary of Region
Isohyetose Lines According to Edwin Duryea Jr. M. Am. Soc. C. E., 1906 30"
Amended Isohyetose Lines 30"
Precipitation Station
Stream-Gauging Station
DRAINAGE AREAS.
WEST SLOPE OF SIERRA NEVADA MTS.
American River above Fair Oaks 1925 Sq. Mi.
Consumnes " " Live Oak Bridge 580 "
Mokelumne " " Clements 639 "
Stanislaus " " Knights Ferry 935 "
Tuolumne " " La Grange 1557 "
Merced " " Merced Falls 1090 "
San Joaquin " " Pollasky 1640 "
Kings " " Red Mountain 1742 "
Kaweah " " Three Rivers 520 "
Tule " " McFarlands 266 "
Kern " " 1st Pt. of Meas'm'nt 2345 "
EAST SLOPE OF SIERRA NEVADA MTS.
Truckee River above Tahoe 519 "
" " " State Line 955 "
" " " Vista 1520 "
Little Truckee " " Boca 166 "
West Carson " " Woodfords 70 "
East " " " Gardnerville — "
West Walker " " Colville 306 "
East " " " Yerington 1100 "
Long Valley Region 444 "
Bishop - Big Pine Region 609 "
Independence Region 357 "
Owens Lake Region 275 "
NOTE
The base for this map is the General Land Office Map of California. Where possible, drainage lines were obtained from the U.S. Geological Survey topographic maps. Listed drainage areas, where differing from those published in Water Supply Papers Nos. 250 and 251, were obtained by careful planimeter measurement from government topographical maps.
MOHAVE DESERT

FIG. 1.

forty tributaries entering at fairly regular intervals from the west. The river terminates in Owens Lake, an alkaline body of water without outlet which disposes of all surplus surface water from the valley by evaporation. Irrigation is necessary for the production of crops, and water is diverted in canal systems from Owens River and tributary streams.

Independence Basin.—The portion of Owens Valley which is the subject of this study will be spoken of as the Independence Basin (Plate I). It lies in the south-central part of the valley, embracing with its tributary drainage area the region between the Sierra Nevada crest and Owens River, lying between Poverty Hills on the north and Alabama Hills on the south. These latter are secondary ranges extending into the valley from the Sierra Nevada and isolating a portion of the valley fill about 25 miles long and from 6 to 8 miles wide.

For the purpose of this study, the surface of the region has been classified as high mountain drainage, intermediate mountain slopes, outwash slope, and valley floor. (Table 1 and Plate I.)

TABLE 1.—Subdivisions of Independence Region.

Subdivision.	Area.		Boundary.		Slope.	Vegetation.	Character of surface.
	Square miles.	Percentage of total.	Upper.	Lower.			
High mountain drainage.	96.0	27	Sierra crest.	Mouth of canyon.	Precipitous to gentle.	Isolated forest trees.	Bare granite and fragmental rock accumulations.
Intermediate mountain slope.	29.4	8	Canyon drainage.	6 500 ft. contour.	2 000 to 3 000 ft. to the mile.	Desert bushes and nut pine.	Fragmental and finely disintegrated rock accumulations.
Outwash slope.	165.3	46	6 500-ft. contour.	Grass land.	300 to 600 ft. to the mile.	Desert bushes.	Boulders, sand and gravel
Valley floor:							
Cultivated....	4.7	1	....	....	Gentle......	Alfalfa, etc....	Soil.
Meadow......	45.1	13	....	....	Gentle to level.	Salt grass, etc.	Soil.
Alkali........	2.7	1	....	....	Level......	None	Soil.
Desert........	13.9	4	....	....	Level......	Desert bushes.	Fine sand.
	357.1	100					

The high mountain drainage embraces the eastern slope of the Sierra Nevada and consists of a series of seventeen small canyons which are the drainage basins of streams tributary to Owens River (Table 2). These canyons are all narrow at the mouth (the 6 500-ft. level) and broaden out more or less toward the summit, presenting a roughly triangular shape. They are separated by high knife-edge ridges, which terminate in triangular slopes facing the valley. They have been cut by water erosion and sculptured by active glaciation above the 7 500-ft. level, their upper portions being well-developed glacial cirques. In many places below the cirques are series of benches occupied by glacial lakes or meadows. Most of the cirque floors are buried beneath morainal accumulations; some of the polished canyon bottoms between the 11 500 and 8 000-ft. levels are swept clean of débris, and others are completely buried by morainal material. Terminal and lateral moraines of considerable size occupy the canyon floors between the 8 000 and 7 000-ft. levels.

TABLE 2.—HIGH MOUNTAIN DRAINAGE AREAS OF INDEPENDENCE REGION.

Creek.	Area.		Elevation.		Shape.	Length of Sierra crest drained, in miles.	Remarks.
	Total, in square miles.	Percentage above 10 000 ft.	Head of canyon, in feet.	Mouth of canyon, in feet.			
Taboose..........	7.16	60	12 000	6 500	Triangular...	3.34	Morainal deposits; regulated run-off.
Goodale..........	4.97	69	12 500	6 500	Triangular...	2.67	Morainal deposits; regulated run-off.
Dry Canyon......	2.48	65	12 000	6 500	Triangular...	1.21	Morainal deposits; no surface run-off.
Division..........	3.88	51	12 000	6 000	Rectangular.	1.88	Morainal deposits.
Sawmill...........	7.64	44	12 000	5 000	Rectangular.	2.75	
Thibaut (N. Fk.).	2.25	20	11 500	6 000	Irregular....	0.0	
Thibaut (S. Fk.)..	2.62	55	12 000	6 000	Irregular....	0.0	
Oak (N. Fk.).....	8.08	65	12 500	6 000	Irregular....	5.17	Morainal deposits; regulated run-off.
Oak (S. Fk).......	7.28	57	12 500	6 000	Irregular....	1.06	
Little Pine........	8.42	74	13 000	6 500	Triangular...	4.60	
Pinyon............	4.29	47	13 000	6 500	Irregular....	1.09	
Symmes..........	4.22	48	13 000	6 300	Triangular...	1.59	
Shepard..........	12.29	66	13 500	6 500	Triangular...	7.95	5.68 miles of crest south of Kings-Kern divide.
Bairs (N. Fk.)....	4.01	43	13 500	6 300	Irregular....	0.0	Lies on east face of Mount Williamson.
Bairs (S. Fk.)....	2.90	41	13 000	6 300	Irregular....	0.0	Lies on east face of Mount Williamson.
George...........	9.10	74	13 500	6 500	Triangular...	3.89	
Hogback..........	4.38	58	13 000	7 000	Irregular....	0.67	
	95.97	55	......	......	....	37.87	

The intermediate mountain slopes (Fig. 2) are the triangular areas terminating the ridges between the canyons, and probably represent the original face of the range before it had been actively eroded (Table 3). Their lower boundary has been arbitrarily placed at the 6 500-ft. contour, and their apexes reach a maximum elevation of about 12 000 ft. They have a steep uniform slope of from 2 000 to 3 000 ft. to the mile, and in general are covered with a mantle of disintegrated rock and slide material which merges into the valley fill.

TABLE 3.—INTERMEDIATE MOUNTAIN SLOPES OF INDEPENDENCE REGION.

Adjoining high mountain drainage areas.	Area, in square miles.	Elevation.			Distance from Sierra crest to center of area, in miles.	Remarks.
		Apex, in feet.	Center of area, in feet.	Lower border, in feet.		
Tinemaha	2.17	(11 000)	8 000	6 500	3.0	
Red Mountain	2.37	(12 000)	8 300	6 500	3.0	
Taboose	3.94	12 200	8 000	6 500	3.3	
Goodale	2.29	11 800	7 200	6 500	2.6	Does not include Dry Canyon.
Division	0.95	9 500	7 500	6 500	3.5	
Sawmill	1.32	10 200	7 500	6 500	3.2	
Thibaut (North Fork)	0.53	10 500	7 500	6 500	3.1	
Thibaut (South Fork)	0.07	7 000	6 700	6 500	4.0	Charlies Canyon yields run-off in normal and above normal years.
Oak (North Fork)	3.62	12 600	7 100	6 500	4.2	
Oak (South Fork)	1.03	10 600	7 100	6 500	3.8	
Little Pine	2.02	11 800	7 400	6 500	3.8	Lime Fork yields run-off in normal and above normal years.
Pinyon	2.89	11 500	7 600	6 500	2.7	
Symmes	0.42	9 200	7 400	6 500	3.2	North Fork similar to Lime Fork.
Shepard	0.97	9 900	7 200	6 500	4.5	
Bairs (North Fork)	0.48	9 100	7 100	6 500	5.0	
Bairs (South Fork)	1.21	10 300	7 800	6 500	4.6	
George	2.09	11 200	8 100	6 500	3.5	
Hogback (one-half)	1.08	10 800	7 900	6 500	4.1	
	29.45	....	....	....	....	

The outwash slope (Fig. 2) is the desert portion of the surface of the valley fill, extending from the 6 500-ft. contour at the base of the Sierra Nevada to the upper edge of grass and irrigated land in the valley (3 900 to 4 000 ft.). Its surface is composed of loose boulders, gravel, and sand, deposited during past ages by torrential streams coming from the mountains. This deposit is of

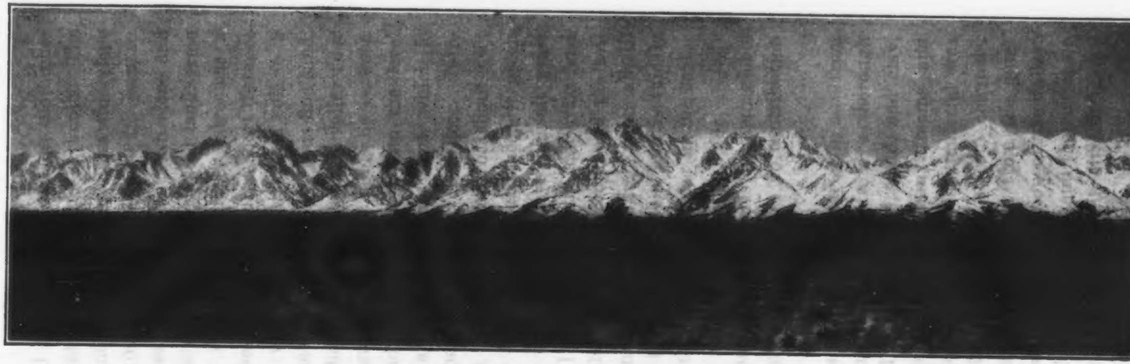


FIG. 2.—EASTERN SLOPE, SIERRA NEVADA.

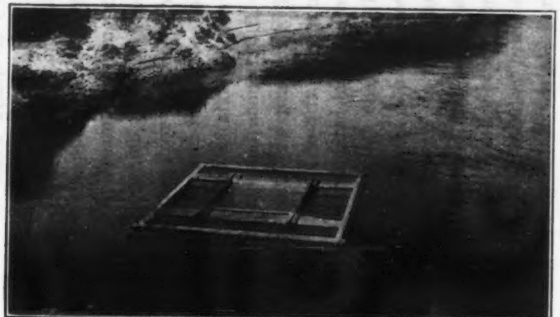


FIG. 3.—EVAPORATION PAN IN OWENS RIVER.

FIG. 4.—EVAPORATION PAN IN SOIL.

unknown depth, and lies on a buried ancient rocky surface, the higher hills of which appear above the present surface as buttes or knolls. The channels of streams draining the mountain canyons cross this slope in trenches, which, near the mountains, are from 25 to 50 ft. deep.

The valley floor embraces the area between the outwash slope and Owens River, and its surface may be classified as irrigated land, grass or meadow land, alkali land, and desert. The upper edge has a maximum slope of about 120 ft. to the mile, but within a short distance it merges into the practically level valley. The surface is soil to a depth of from 1 to 3 ft., except on the desert land, where it is fine sand. Most of the irrigated land is along the upper margin of the valley floor adjoining the creek channels. The grass or meadow lands lie between and to the east of the ranches, and extend well out into the level valley. The growth is most luxuriant in the spring zone, which is about ½ mile wide and is at the upper edge of the valley floor. Here are numerous small flowing springs, with temperature of about 62°, which start the meadow grass early in the season and keep it green until late in the autumn. Farther out in the valley, the salt grass makes a green carpet from May until late July. In the salt-grass land there is always a deposit of alkali about the plant roots, and the soil surface is crusted. The spring zone, however, is free from alkali. The worst alkali land is practically bare of vegetation and is thickly crusted with white salts. It lies in the more level areas in the center of the valley.

The desert area to the east of Owens River yields no appreciable run-off, and, owing to its light precipitation, it makes no contribution to the ground-water.

The alluvial material which forms the valley fill varies in size from large boulders to fine clay, and, in arrangement, from a thorough mixture of all sizes to layers of well-assorted gravel, sand, and clay. The transporting medium was water, both mountain streams and Owens River taking part in the work. Some of the material was deposited in the beds and on the sides of shifting stream channels, and much of the finer sand and clay was deposited from the quiet waters of a large lake which occupied the lower portion of the valley. The structure of the valley fill, therefore, is complex, and the character of the alluvial material underlying a given locality is difficult to determine without actual examination from borings.

A number of borings, ranging in depth from 250 to 500 ft., have been made in the basin by the City of Los Angeles, in connection with the development of the aqueduct supply. In general, the materials encountered were clay, sand, and coarse gravel in layers varying in thickness from a few inches to 150 ft. Coarse material in thin layers interbedded with clay predominates along the upper edge of the valley floor in the spring belt. All wells in this belt yield Artesian flows of from 1 to 2 sec-ft. The material is progressively finer and occurs in thicker strata east of this belt, toward the center of the valley, and the Artesian flows decrease in volume. Near Owens River, fine sand and clay in alternate layers is the only material encountered above the 300-ft. depth.

The streams from the Sierra Nevada were by far the most active in the work of building up the valley fill. Their loads were acquired in the mountain canyons and carried out into the valley, where they were dropped in order of size as the velocity of flow decreased. The old lake level stood at an elevation of about 3 790 ft. for a long period, as shown by beach lines on the east slope of the Alabama Hills. The present 3 790-ft. contour lies near the spring and Artesian belt. The finer materials between the spring belt and Owens River are evidently lake deposits. The ancient lake was contemporaneous with other geologic lakes of the Great Basin, such as Lakes Bonneville and Lahontan. The geologic history of these lakes shows many wide fluctuations of water level, covering long periods of time. The interbedding of fine and coarse material encountered in the spring belt is evidently the result of such fluctuation, as the sudden checking of the velocity of a stream on entering a body of still water results in the immediate deposition of coarse material. The Artesian and spring conditions, therefore, result from hydrostatic pressure on the water entrapped in these wedges of coarse material.

Two cross-sections of the valley, showing the probable geologic structure, were constructed along the Thibaut and Independence Sections (Fig. 5). The topography for these sections was obtained from the U. S. Geological Survey's map of the Mount Whitney quadrangle, and the character of the surface material was determined by field inspection. The exposed slopes of bed-rock on each side of the valley were joined beneath the valley floor, and the arrangement of the material filling the basin thus formed was represented according to

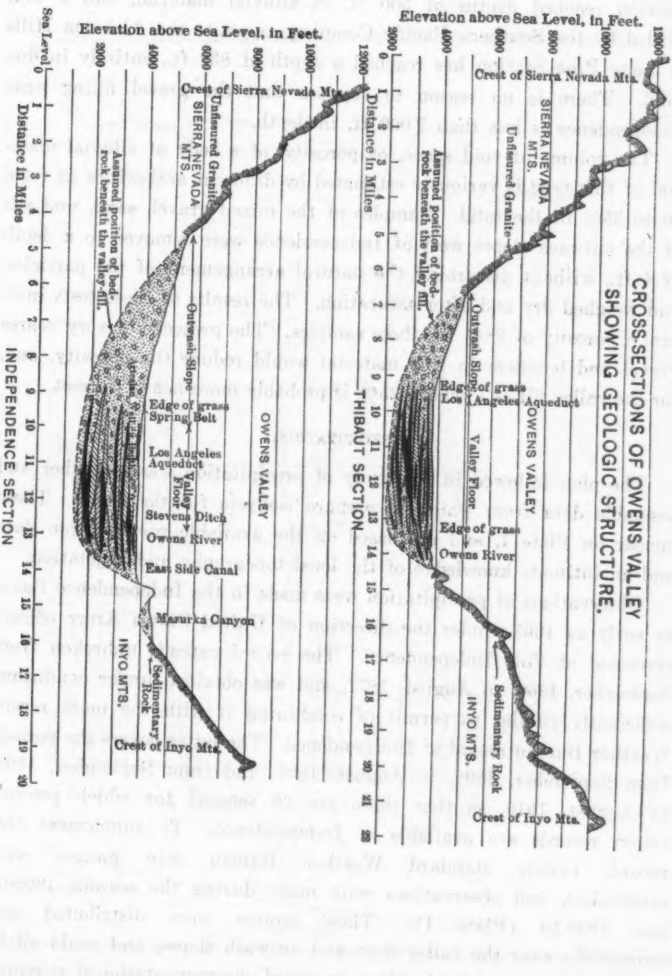

CROSS-SECTIONS OF OWENS VALLEY
SHOWING GEOLOGIC STRUCTURE
Elevation above Sea Level, in Feet.
Crest of Sierra Nevada Mts.
SIERRA NEVADA MTS.
Unfissured Granite
Assumed position of bed-rock beneath the valley fill
Distance in Miles
Outwash Slope
Edge of grass
Los Angeles Aqueduct
OWENS VALLEY
Valley Floor
Edge of grass
Owens River
Sedimentary Rock
INYO MTS.
Crest of Inyo Mts.
THIBAUT SECTION
Elevation above Sea Level, in Feet.
Sea Level
Crest of Sierra Nevada Mts.
SIERRA NEVADA MTS.
Unfissured Granite
Assumed position of bed-rock beneath the valley-fill
Outwash Slope
Edge of grass
Spring Belt
Los Angeles Aqueduct
Valley Floor
Stevens Ditch
Owens River
East Side Canal
Mazurka Canyon
OWENS VALLEY
INYO MTS.
Sedimentary Rock
Crest of Inyo Mts.
Distance in Miles
INDEPENDENCE SECTION

FIG. 5.

the best available knowledge, the strata of fine material being indicated by solid black. On the diagram, the greatest depth of alluvial filling measures 2 500 ft. in the Independence Section, and 1 800 ft. in the Thibaut Section. Two of the aqueduct wells near the Independence Section reached depths of 500 ft. in alluvial material, and a well drilled by the Southern Pacific Company, opposite the Alabama Hills at Lone Pine Station has reached a depth of 832 ft., entirely in fine sand. There is no reason to suppose that the gravel filling near Independence is less than 2 000 ft. in depth.

The volume of void space, or porosity, of a body of alluvial material of this type is variously estimated by different authorities at from 20 to 35% of the total. Samples of the mixed gravel, sand, and silt of the outwash slopes west of Independence were removed to a depth of 4 ft., without disturbing the natural arrangement of the particles, and weighed dry and after saturation. The results of these tests indicate a porosity of 28% for these samples. The presence of very coarse gravel and boulders in this material would reduce the porosity, and, for the valley fill as a whole, 25% is probably more nearly correct.

Precipitation.

The plan followed in the study of precipitation was to gather and assemble data from which to prepare isohyets for the basin. These appear on Plate I, and are based on the available precipitation data and an intimate knowledge of the local topography and vegetation.

Observations of precipitation were made in the Independence Basin as early as 1865, under the direction of United States Army officers stationed at Fort Independence. The record extends unbroken from September, 1866, to August, 1877, and was obtained under conditions sufficiently similar to permit of combining it with the more recent Weather Bureau record at Independence. The latter covers the periods from September, 1892, to August, 1895, and from September, 1898, to August, 1910, so that there are 26 seasons for which precipitation records are available at Independence. To supplement this record, twenty standard Weather Bureau rain gauges were established, and observations were made during the seasons 1908-09 and 1909-10 (Plate I). These gauges were distributed systematically over the valley floor and outwash slopes, and could all be reached during one day by three mounted observers stationed at points


MAP OF INDEPENDENCE REGION
SHOWING
DRAINAGE AREAS, STREAM CHANNELS
AND PRECIPITATION STATIONS.
SCALE OF MILES
1.0 0.5 0 1 2 3 4 5
LEGEND.
Boundary of Mountain Drainage Areas
Boundary of Region
Isohyetose Lines
Stream Gauging Station Perennial Flow
Stream Gauging Station Intermittent Flow
Precipitation Station
West Boundary Valley Floor
BASE OF INYO MTS.
SIERRA NEVADA MTS.
LINE OF
VALLEY FLOOR
Owens River
Los Angeles Aqueduct
Poverty Hills
Red Mountain
Charlies Butte
Alabama Hills
Independence
Kings River Drainage
Kern River Drainage
Kings-Kern Divide
SOUTHERN PACIFIC SYSTEM (NEVADA AND CALIFORNIA R.R.) (NARROW GAUGE)

in the valley where shelter was available. Four records were also available in Owens Valley, outside of the Independence Basin, at Bishop, Lone Pine, Laws, and Keeler. The Bishop and Lone Pine records are kept by co-operative Weather Bureau observers, and cover 15 and 5 years, respectively. The Laws and Keeler records are kept by railroad agents, and are for 13 and 24 years.

The distribution of total precipitation, with respect to geographic location, in the Independence Basin and adjoining areas depends to a great extent on topographic features, notably mountain ranges and valleys, although a consistent variation is also evident with changes in latitude. The controlling topographic feature is the Sierra Nevada, which has a general northwest and southeast trend.

This relation of precipitation and topography is well shown by studying observations made along cross-sections of the Sierra Nevada laid out at right angles to the trend of the range. Two such sections are indicated on Fig. 1 as the Central Pacific and Mokelumne Sections. The relations of mean annual precipitation, altitude, topographic position, and profiles of ground surface are presented graphically for the two sections in Diagrams 1 to 6 of Plate II. The marked similarity in the curves for the two sections indicates that the quantity of precipitation at points in a transverse section of the range conforms to some general law. Elevation, obviously, is not the controlling factor, for above the 5 000-ft. level the precipitation decreases with increase in altitude. The slope of the ground surface appears to be the most important element involved, as is seen from Diagrams 2, 3, 5, and 6 of Plate II. The phenomenon results from the condensation of aqueous vapor due to adiabatic cooling of masses of moist air driven up the slope of a mountain range by the prevailing winds. The region of maximum precipitation is at the lower cloud limit on the windward slope of the range, and above this the latent heat liberated by condensation raises the temperature above the dew point, resulting in decreased precipitation. After crossing the summit of a high range, the descending mass of air contracts in volume, thereby raising the temperature rapidly above the dew point and resulting in marked decrease of precipitation.

The increase of precipitation with elevation was first observed by Mr. S. A. Hill,* in studying rainfall in the northwest Himalayas of

* "California Hydrography," by J. B. Lippincott, M. Am. Soc. C. E., Water Supply Paper No. 81, U. S. Geological Survey, p. 354.

India, and he developed for that region the empirical formula, $R = 1 + 1.92h - 0.40h^2 + 0.02h^3$, in which R represents the quantity of rain and h the relative height, in units of 1 000 ft., above an assumed plane which is itself 1 000 ft. above sea level. This equation, when platted, gives a curve very similar to that shown in Diagrams 1 and 4 of Plate II, the plane of maximum rainfall being 4 160 ft. above sea level. The equation does not apply to conditions on the leeward slope of a range, however, to judge by the discontinuity at the crest line shown on the Sierra Nevada curves. The straight-line relation between precipitation and elevation, which is often assumed in engineering computations, thus appears to have a very limited use, and to be at best a rough approximation.

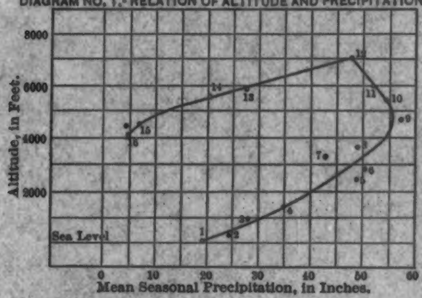
The Los Angeles Aqueduct precipitation stations in Owens Valley lie in three groups, indicated on Fig. 1 and Plate I, as the Taboose, Oak, and Bairs Sections. The 2-year records for these stations were reduced to averages by comparison with the 26-year record at Independence. The platted curves for these sections (Plate II) are all similar in shape, and agree with the desert slope portion of the Central Pacific and Mokelumne curves. The highest point on each of the Owens Valley curves was obtained from the measured run-off of the canyons crossed by the section and a run-off factor chosen after careful study of precipitation and run-off data for Kings River, which drains the slope of the Sierra Nevada to the west. The precipitation at the mouth of the canyons was known from the rain gauge observations, and the average precipitation over each canyon from the run-off. Most of these canyon drainage areas are isosceles triangles in plan, the base being along the crest of the mountains. Also, judging by the Central Pacific and Mokelumne Sections, the precipitation increases uniformly from the base of the mountains on the desert side to the summit. Hence, the precipitation at the center of area of each drainage basin is equal to the average precipitation over the whole area, and the precipitation at the summit is obtainable by a simple proportion.

Preliminary to the establishment of isohyets or lines of equal annual precipitation for the Independence Basin, a broad study of precipitation was made for the whole Sierra Nevada range from Lake Tahoe to the Mojave Desert (Fig. 1). This was based on the California Water and Forest Association rainfall map of the State, pre-

DIAGRAMS SHOWING THE
INFLUENCE OF ALTITUDE AND TOPOGRAPHIC LOCATION ON PRECIPITATION.
AS ILLUSTRATED BY THE SIERRA NEVADA MTS.

CENTRAL PACIFIC GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 1.- RELATION OF ALTITUDE AND PRECIPITATION.



NOTES

Observations on Central Pacific and Mokelumne Groups of gauges by U.S. Weather Bureau.

Observations on Taboose, Oak, and Bairs Groups of gauges by City of Los Angeles. (See Tables 16 and 17.) For location of gauges see Drainage Map of Sierra Nevada Mts. and Drainage Map of Independence Region.

The Season is from Sept. 1st to Aug. 31st.

DIAGRAM NO. 2.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

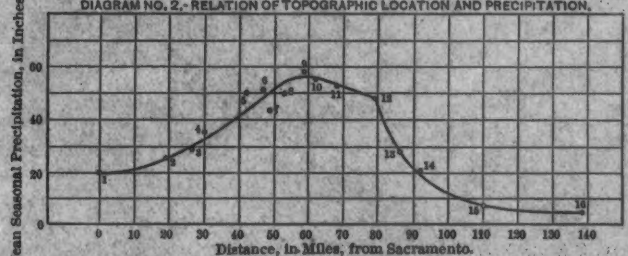
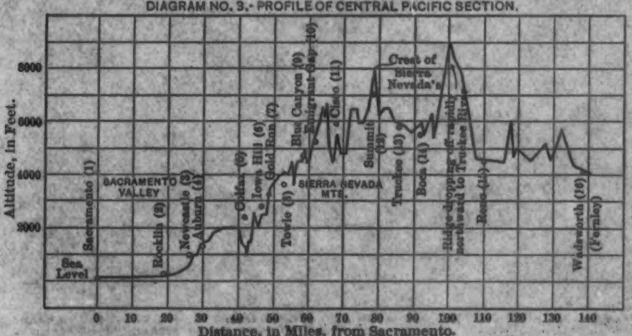


DIAGRAM NO. 3.- PROFILE OF CENTRAL PACIFIC SECTION.



MOKELUMNE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 4.- RELATION OF ALTITUDE AND PRECIPITATION.

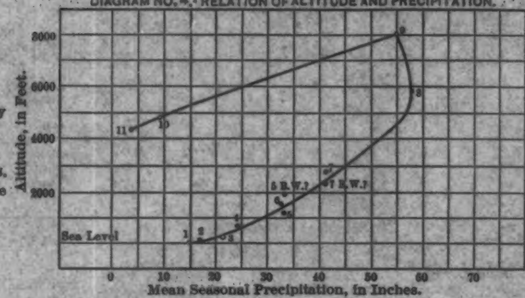


DIAGRAM NO. 5.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

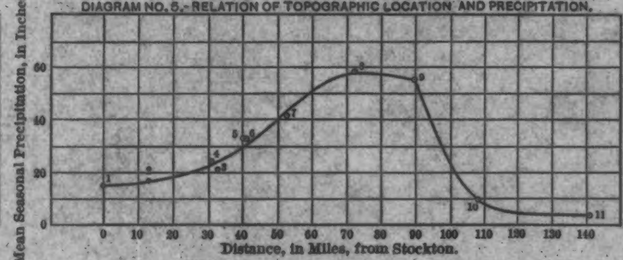
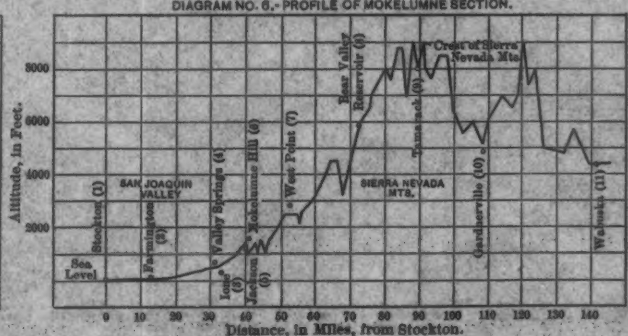


DIAGRAM NO. 6.- PROFILE OF MOKELUMNE SECTION.



TABOOSE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 7.- RELATION OF ALTITUDE AND PRECIPITATION.

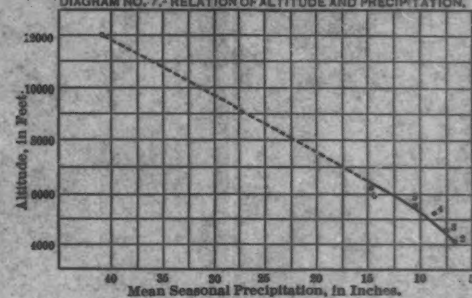


DIAGRAM NO. 8.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

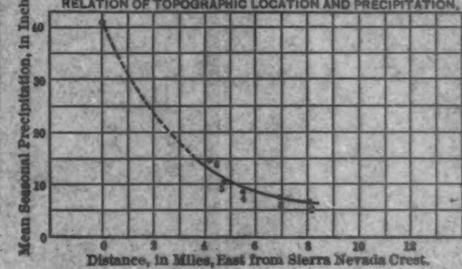
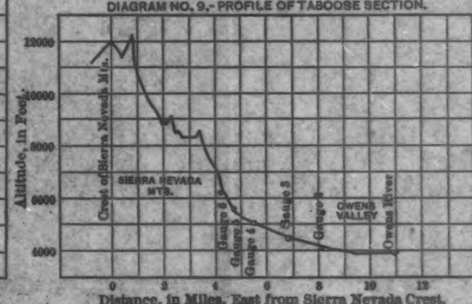
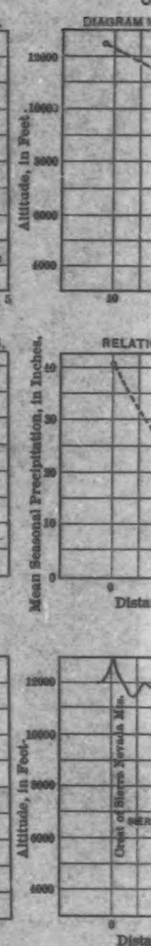


DIAGRAM NO. 9.- PROFILE OF TABOOSE SECTION.



OA

DIAGRAM NO



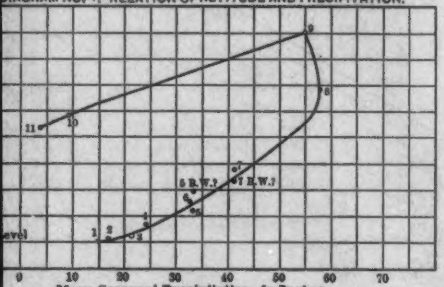
DIAGRAMS SHOWING THE

INFLUENCE OF ALTITUDE AND TOPOGRAPHIC LOCATION ON PRECIPITATION.

AS ILLUSTRATED BY THE SIERRA NEVADA MTS.

MOKELUMNE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 4.- RELATION OF ALTITUDE AND PRECIPITATION.



NO. 5.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

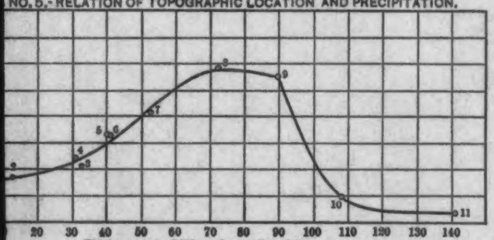
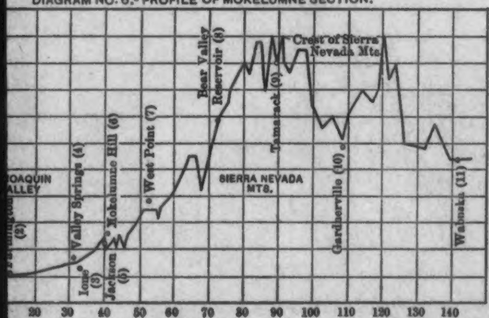


DIAGRAM NO. 6.- PROFILE OF MOKELUMNE SECTION.



TABOOSE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 7.- RELATION OF ALTITUDE AND PRECIPITATION.

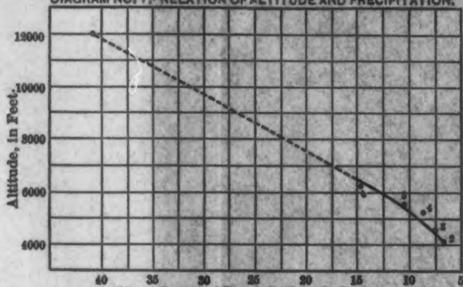


DIAGRAM NO. 8.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

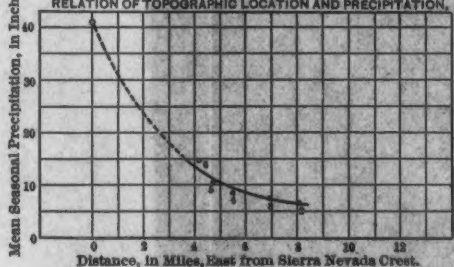
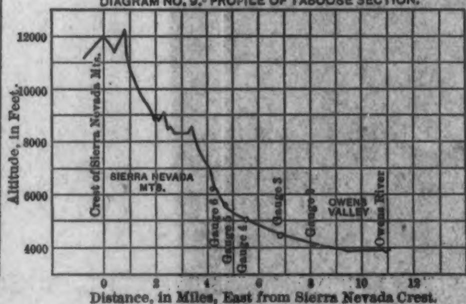


DIAGRAM NO. 9.- PROFILE OF TABOOSE SECTION.



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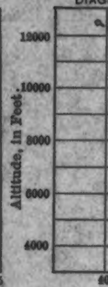

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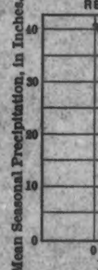
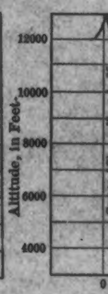




OAK GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 10.- RELATION OF ALTITUDE AND PRECIPITATION.

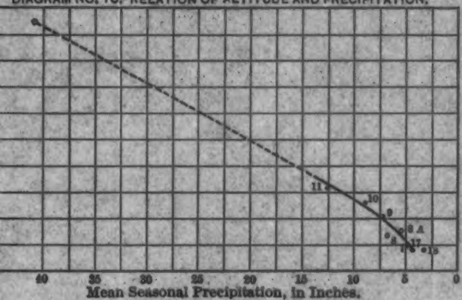


DIAGRAM NO. 11.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

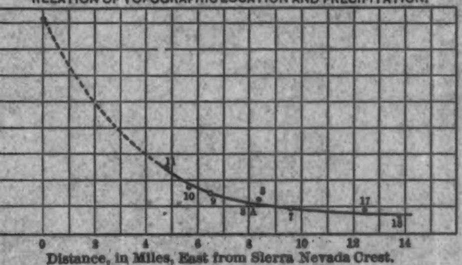
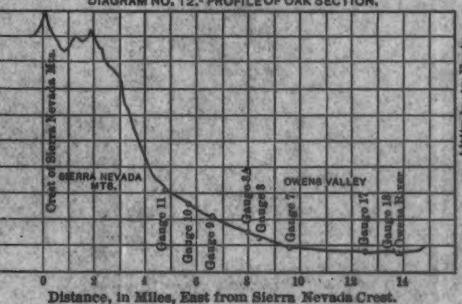


DIAGRAM NO. 12.- PROFILE OF OAK SECTION.



BAIRS GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 13.- RELATION OF ALTITUDE AND PRECIPITATION.

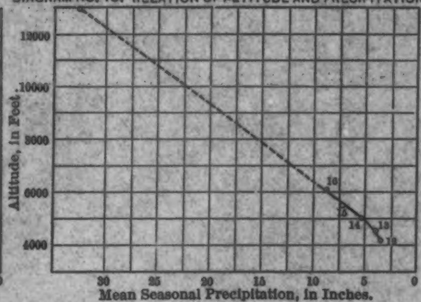


DIAGRAM NO. 14.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

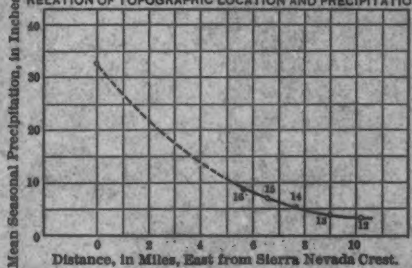
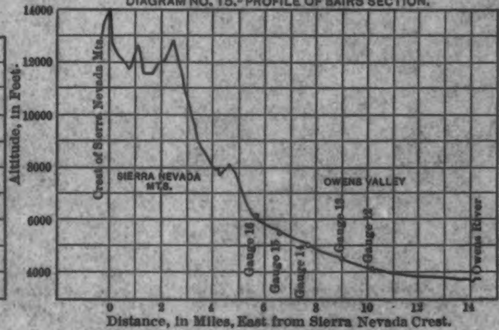


DIAGRAM NO. 15.- PROFILE OF BAIRS SECTION.



pared in 1900, as revised between American and Kings Rivers by Edwin Duryea, Jr., M. Am. Soc. C. E., in 1908. With all available rainfall data to date, including the Los Angeles Aqueduct and several private records, the writer has made further revisions over the Southern Sierra and Owens Valley. The Water and Forest Association isohyets, as amended by Mr. Duryea, appear on Fig. 1 as solid lines, and revisions proposed by the writer are represented by dotted lines. Isohyets are shown with greater detail for the Independence Basin on Plate I.

Stream Flow.

Stream flow data essential to the determination of inflow and outflow for an underground reservoir, are the run-off from precipitation, the seepage from or into stream channels, and the flow of springs. Precipitation which finds its way into surface streams without absorption may, along some portion of the channel, percolate into porous gravels and join the subterranean supply. On the other hand, water may escape from the underground reservoir by seepage into stream channels where the general water plane is at a higher elevation than mean water level in the stream. Escape may also occur from springs.

The problem in the Independence Basin was, first, to classify the surface as to run-off characteristics and determine the run-off from each subdivision; second, to ascertain seepage losses from the seventeen tributary mountain streams between the canyon mouths and the valley floor; third, to determine the flow of springs which represent water escaping from the basin; and fourth, to ascertain whether Owens River made or lost water in passing through the basin.

Run-off.—It was early observed that the run-off characteristics of the four areas into which the region was classified for study (Table 1) were similar.

The clay soils of the valley floor occasionally yield a small run-off during and following winter precipitations of 1 in. or more in 24 hours, or warm rain falling on old snow. This water gathers and passes off into Owens River within a few hours by way of four waste channels. A study of the available data shows that the average total run-off from precipitation on the valley floor is about 2 sec-ft. of continuous flow.

The outwash slopes yield no appreciable surface run-off, on account of the porous gravel formation and the great depth to ground-

water. This fact has been established by repeated observations during and after rainstorms and thaws, and is confirmed by the noticeable absence of recent drainage channels or washes, except those of streams which derive their water from high mountain drainage areas.

The intermediate mountain slopes yield a small run-off during May and June, when the temperature at that level is sufficient to melt the accumulated winter snow, but the small streams do not advance far over the outwash slopes before they are entirely absorbed. If the precipitation of the preceding winter is below normal, the snow melts before the hot weather comes, and is absorbed at once. Springs are common along the lower borders of these slopes, the source being the melted snow absorbed by the porous material above and brought to the surface where it comes into contact with impervious formations. In only a few places does such water find its way into living streams.

The high mountain drainage areas have an abundant run-off, and perennial streams flow from all but one of them. The source of this water is precipitation, in the form of snow and rain, which falls within the drainage areas, and to a small extent snow dust carried over the summits by the prevailing west and northwest winds of winter and spring. For all practical purposes, the average discharge at the mouth of the canyon represents the average precipitation within the drainage area minus losses by evaporation from exposed snow surfaces. The underflow from these areas is negligible.

Stream discharge from the canyons is at a minimum from September to April. The flow during these months is remarkably uniform, and is entirely uninfluenced by the current storms, though from 70 to 80% of the annual precipitation occurs between November 1st and March 31st. The low-water flow is derived from springs and from the slow melting of the snow layer exposed to the earth's latent heat. Streams are usually frozen over by November, and as late as April they flow nearly to the mouths of the canyons in tunnels under the snow. Between April 1st and 20th air temperatures increase sufficiently to melt the snow at the lower elevations, and the streams begin to rise. There is an increase in air temperatures and stream flow from this date until the maximum flood crest is reached, some time between June 15th and July 15th, depending on the quantity of snow to be melted. Stream flow then decreases until some time in September,

after which low water prevails. About 70% of the annual run-off of the streams occurs during May, June, July, and August.

Percolation from Stream Channels.—The United States Geological Survey gauging stations on streams draining the high mountain areas are at the lower edges of the outwash slopes, just above the division boxes which apportion the water for use on the ranches of the valley floor. After leaving its canyon each stream traverses several miles of channel before reaching the gauging station, and preliminary observations in June, 1908, showed that considerable water (in some streams 50%) disappeared between the two points. It was necessary, therefore, either to establish regular gauging stations at the canyon mouths and depend on records for short periods, or to devise some means of computing the run-off from the high mountain areas from the existing Government records, which extended over 6 years. The latter method was chosen, and the results have proved very satisfactory.

The loss from these stream channels occurs as percolation into the porous alluvial material, direct evaporation from water surface, and transpiration from vegetation growing along the stream borders. Evaporation and transpiration losses were too small to be detected in current-meter work. As the expense of installing and maintaining weirs was prohibitive, the problem resolved itself into a study of percolation from stream channels.

There are three factors to be considered in a study of the subject: the rate of percolation, the area through which percolation occurs (the wetted perimeter), and the period of time during which a given unit of water is exposed (velocity of flow). The rate of percolation depends on (1) the character of the channel lining and the medium surrounding the channel, as regards size of pores and porosity; (2) the pressure gradient, depending on the difference in level of the surface of the water in the channel and the ground-water surface; and (3) the temperature of the water.

The effect of an increase in the wetted perimeter, other conditions being the same, is obviously to increase the percolation, but such change is accompanied by a proportionally larger increase in the velocity of flow, which reduces the time of exposure of a given volume of water. The net result, considering the total flow, is, therefore, a proportionally smaller percolation, although this effect may be counter-

acted to a certain extent by the scouring of a non-porous channel lining due to the increased carrying power of the stream. The whole matter is affected by so many indeterminate conditions that a general mathematical analysis is impossible, but, with these ideas in view, a study was made of each channel within the ordinary range of temperature and discharge.

The field work consisted of making comparative current-meter measurements at upper and lower stations on each creek, giving proper allowance of time for the passage of water between the two points. The measurements were made at intervals of from 6 weeks to 2 months, and extended over the period from June 15th, 1908, to September 15th, 1909, including the high-water periods of wet and dry seasons. Gauging sections were prepared at the mouth of the canyon on each creek. Estimates of the time required for the passage of water between stations were based on actual trial with aniline dye. Very little fluctuation in discharge was observed in any of the creeks between 8 A. M. and 5 P. M., even in the high-water period.

The temperature of the water as it issues from the canyons varies from 35° to 42° Fahr., in winter, and from 48° to 53° Fahr., in summer. In winter the temperature does not increase much as the water travels toward the valley. After leaving the protecting cover of the snow in the canyons during December and January the water actually becomes colder, ice prevailing for several weeks. In summer there is an average increase of 10° between the stations at the mouth of the canyon and in the valley.

Several methods were attempted for generalizing the results, but the most satisfactory was a graphical one, in which losses were platted as abscissas and stream discharges as ordinates with rectangular coordinates (Plate III).

It was found that for each channel a straight line expressed the relation of these two quantities from April to October, inclusive. During the remaining 5 months, the relation is not clear, but the total loss is then so small that it can be obtained by inspection without affecting the accuracy of the computed discharge at the mouth of the canyon. Total losses are platted on the basis of discharge, both at lower and upper stations, so that, in correcting the Government records to obtain the true yield from high mountain drainage areas, the quantity desired can be obtained at once by entering on the X-axis

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
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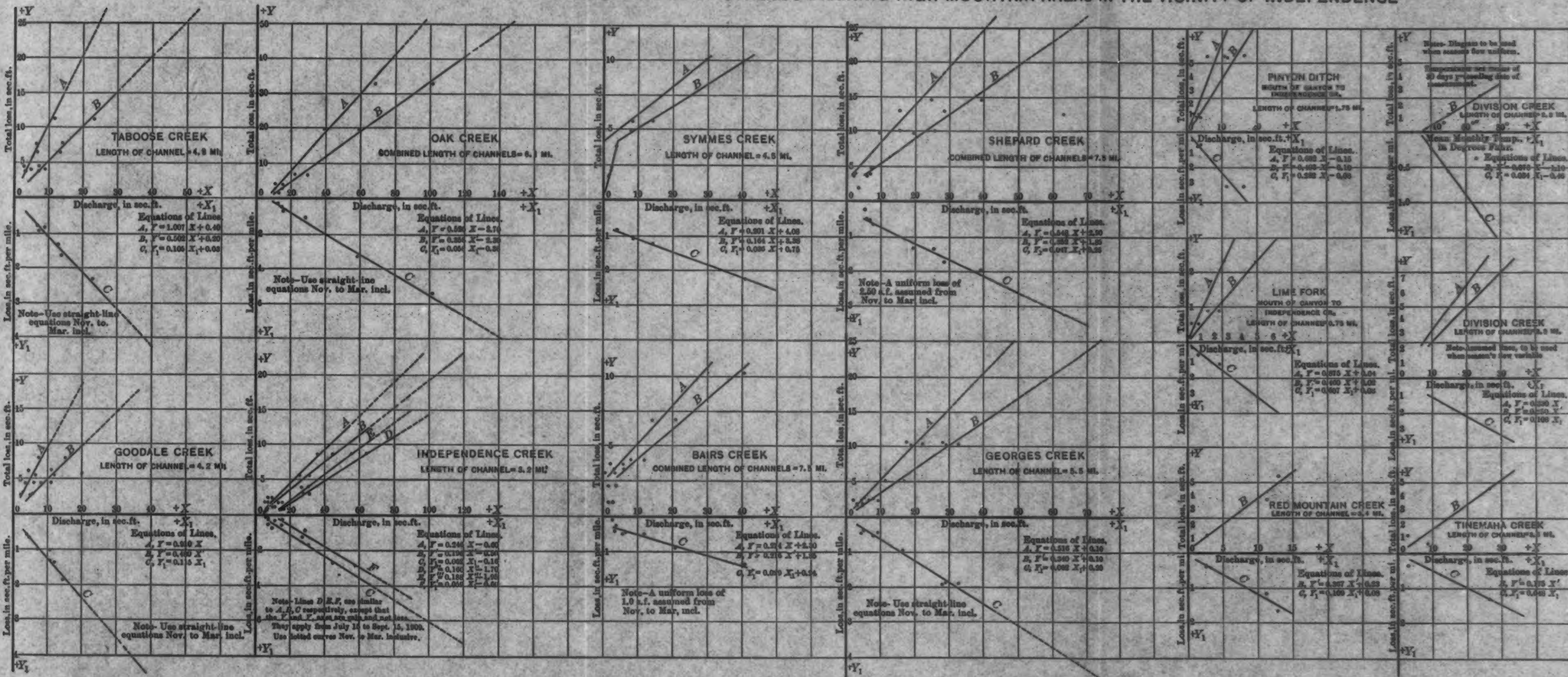
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.

SEEPAGE LOSS DIAGRAMS FOR OUTWASH SLOPE CHANNELS OF STREAMS DRAINING HIGH MOUNTAIN AREAS IN THE VICINITY OF INDEPENDENCE



the discharge at the lower or Government station. It is also possible at the same time to read the loss, in second-feet to the mile, from the straight line below the X-axis.

The diagrams on Plate III were prepared for the purpose of computing (1) seepage losses above the U. S. Geological Survey gauging stations, and (2) the actual yield of high mountain drainage areas. In obtaining the field data, discharge measurements were made with small Price current meters during the period, June to August, 1909, inclusive, covering a season of small run-off and one of large run-off. The accuracy is up to the standard for a stream of this type. Measurements of medium and high stages were difficult, on account of the rough sections and very high velocities. The upper gauging station is at the mouth of the canyon, or below impervious dikes forcing seepage water to the surface. The average elevation is 6 000 ft. The lower station is that used by the U. S. Geological Survey (unless otherwise noted), which is above the diversion and has an average elevation of 4 100 ft. Below the latter there is ½ mile or more of channel suffering a large seepage loss which is not included. The length of the channel was obtained by scaling from the Mt. Whitney Quadrangle, and, for Taboose, Red Mountain, and Tinemaha Creeks, from a triangulation survey. Elevations were obtained in a similar manner. In using the diagrams, the straight-line relation applies from April to October, inclusive, during which time temperature and discharge vary similarly. The dotted portions are not supported by field data, and are to be used with judgment. From November to March, inclusive, temperature is the only effective variable, and arbitrary values for loss have been selected. Line "A" expresses the relation of total loss to discharge at the U. S. Geological Survey Station; Line "B" expresses the same relation at the mouth of the canyon; and Line "C" expresses the loss per mile to discharge at the mouth of the canyon. The diagrams are arranged so that all the four values involved may be obtained by entering any one of them.

There are some interesting conclusions to be drawn from these diagrams. In general, the quantity of water percolating from the channels studied varies with the time of year and with different channel conditions. Variation with the time of year is due to the combined effects of temperature, area of wetted perimeter, and velocity of flow. These work more or less in harmony during April to October, inclusive,

and produce the straight-line relation of total loss and discharge. From November to March, inclusive, canyon discharges remain practically constant, showing that variations are largely controlled by temperature. Discharges are then so small, however, that errors of measurement are appreciable, and losses by evaporation have greater weight, so that the true relation of loss and temperature does not appear. A possible relation between total loss and temperature is suggested by the results on Division Creek, where the discharge at the mouth of the canyon was practically uniform during the period of study. Using as ordinates the mean air temperature at Independence for the 30 days preceding each date of measurement, we obtain a straight line which crosses the X-axis at about 35 degrees. A line supported by additional data might cross nearer 32°, the temperature at which percolation becomes physically impossible.

The character of the surrounding medium was the only channel condition which noticeably affected percolation. The loss from a channel crossing fissured lava, even where the lava was covered by a thin sheet of alluvium, was 30% greater than that in coarse alluvium. The streams studied do not overflow their channels, so that the effect of varying channel slopes and wetted perimeter could not be studied.

The run-off from each mountain canyon was computed from U. S. Geological Survey monthly mean discharge records, by use of the diagrams. The results are summarized in Tables 4 and 5. The missing seasons were estimated from the unbroken records of neighboring streams after a detailed study of yield per square mile and annual departure from normal for each stream. The long-term mean discharge was obtained by comparison with the Kings River record, which covered 21 years. This stream was chosen because its drainage area adjoined most of the Owens Valley streams and because conditions affecting run-off were more nearly similar than on any other stream. The results indicate that the total annual mountain run-off during the period of record was 153 annual sec-ft. and the normal 130 annual sec-ft. This total does not include Red Mountain and Tinemaha Creeks, which pass out of the basin after crossing a portion of the outwash slope. The normal run-off per square mile for streams north of the Kings-Kern divide with 60% or more of area above 10 000 ft., is 1.75 sec-ft., and, for streams similarly situated, with less than 60% above 10 000 ft., 1.18. For streams south of the Kings-Kern divide,

TABLE 4.—SEASONAL DISCHARGE, IN SECOND-FEET, AT UNITED STATES GEOLOGICAL SURVEY STATIONS, OF CREEKS TRIBUTARY TO INDEPENDENCE REGION.

(Figures in parentheses are estimated.)

Creek.	Year Beginning September 1st.						Observed 5-year mean.	Computed 21-year mean.
	1904.	1905.	1906.	1907.	1908.	1909.		
Taboose	(5.7)	11.4	10.3	4.7	8.6	5.9	8.2	6.5
Goodale	(3.9)	5.4	6.6	3.8	6.4	5.3	5.5	4.3
Dry Canyon	0	0	0	0.0	0	0	0	0
Division	(3.9)	7.2	10.9	7.6	9.7	9.9	9.1	7.2
Sawmill	....	5.4	(7.6)	(5.0)	7.3	7.2	6.5	5.1
Thibaut	(0.5)	(1.0)	(1.0)	0.8	0.9	0.2	0.6	0.5
Oak	(13.0)	31.8	23.9	15.8	30.8	18.4	24.1	19.0
Little Pine } Pinyon }	(10.7)	28.5	22.5	11.8	25.8	17.2	21.2	16.7
Symmes	....	(2.8)	3.1	0.8	6.3	1.1	2.8	2.2
Shepard	....	23.1	11.0	7.2	12.9	7.7	12.3	9.8
Bairs	....	8.0	4.8	2.0	6.1	2.4	4.7	3.7
George	....	18.6	10.9	6.5	13.0	6.8	11.2	8.8
Hogback	(0)	(1.0)	(0.5)	(0)	(0.5)	0	0.4	0.3
	....	144.2	113.1	66.0	128.3	82.1	106.7	84.0
Percentage of totals at mouth of canyon	....	68	63	61	66	63	65	65

TABLE 5.—SEASONAL DISCHARGE, IN SECOND-FEET, AT MOUTH OF CANYON, OF CREEKS TRIBUTARY TO INDEPENDENCE REGION.

(Figures in parentheses are estimated.)

Creek.	Year Beginning September 1st.						Observed 6-year mean.	Computed 21-year mean.
	1904.	1905.	1906.	1907.	1908.	1909.		
Taboose	11.9	19.2	20.4	9.8	16.0	12.4	15.0	12.7
Goodale	7.6	9.9	12.3	7.2	11.2	10.2	9.8	8.3
Dry Canyon	(3.9)	(5.1)	(6.3)	(3.7)	(5.7)	(5.2)	5.0	4.2
Division	(4.4)	6.9	11.2	6.9	8.7	9.5	7.9	6.7
Sawmill	(4.0)	6.5	9.1	5.5	7.3	7.2	6.6	5.6
Thibaut	(1.9)	(3.1)	(4.4)	(2.7)	(3.5)	(3.4)	3.2	2.7
Oak	16.7	43.2	33.8	21.2	42.6	25.0	30.4	25.8
Little Pine	10.7	24.6	20.5	12.0	24.3	16.7	18.1	15.3
Pinyon	(3.6)	(8.9)	(6.6)	(4.2)	9.2	3.5	6.0	4.8
Symmes	(3.5)	(8.8)	7.0	3.7	10.2	4.9	6.2	5.3
Shepard	(11.1)	31.4	18.9	13.5	2.09	14.1	18.3	15.5
Bairs	(4.3)	11.8	7.7	4.2	9.4	4.8	7.0	5.9
George	(8.2)	23.8	16.5	9.8	18.7	10.3	14.6	12.4
Hogback	(2.8)	(7.8)	(5.7)	(4.2)	(6.8)	(3.7)	5.2	4.4
	94.6	211.0	180.4	108.6	194.5	130.9	153.3	129.6
Percentage reaching U. S. G. S. Stations.	....	68	63	61	66	63	65	65

the normal run-off is 1.36 and 0.86 sec-ft., respectively. It is also of interest to note that only 65% of this run-off reaches the Government gauging stations.

Springs.—The occurrence of springs in the basin is due to the reappearance of water which originally fell within its boundaries as precipitation and was absorbed. There are, in general, three types of springs which give rise to surface streams: those which derive their supply from precipitation on the intermediate mountain slopes, and appear at the base of these slopes; those which derive their supply from precipitation and stream percolation, and appear along the upper edge of the grass land; those which derive their supply from precipitation on lava flows, and appear at the lower borders of the flows.

The springs of the first type are not deep seated; they represent the drainage from the superficial deposits lying on the triangular mountain slopes between canyons. The temperature of their water is about 47° or 48° Fahr., and the flow in many of them increases in early summer and decreases during late summer and autumn. The water from most of these springs sinks into the porous gravels of the outwash slope, and joins the main body of ground-water in the basins.

The line of springs along the upper edge of the grass land represents the intersection of the natural surface of the ground and the surface of the ground-water. The water has penetrated rather deeply into the gravel fill, and issues with a temperature of about 62° Fahr., which is 5° higher than the mean annual temperature at Independence and 1° lower than that of water flowing from Artesian wells in the same location. The flow of these springs is variable, being least in late summer and greatest in early spring, with regular fluctuation between these dates, evidently depending on ground-water stages within the grass area. Only during the winter months is the discharge sufficient to be the source of surface streams which flow any considerable distance, and even then there are only a few of such streams which reach Owens River. Most of the yield of these springs is lost by evaporation and transpiration. The winter discharge of individual springs varies from 0.5 sec-ft. down to a quantity which is only enough to fill small pools of standing water, from which the evaporation equals the yield. The total winter discharge from all these springs is about 4 sec-ft.

The springs issuing from the lava formations are unique in having uniform discharges throughout the year and a temperature of 57° Fahr. The water is probably derived from precipitation on the lava surface, absorbed by the porous rock, and, by reason of the peculiar formation, gathered and delivered at the lower margin of the flow. The largest of these is Blackrock Springs (Plate I), 9 miles north of Independence. It has a discharge of 23 sec-ft., which flows out across the valley floor in two sloughs, each emptying into a series of shallow lakes. From November to March, inclusive, an average flow of about 7 sec-ft. reaches Owens River, but during the remainder of the year all the water is lost by seepage, evaporation, and transpiration. Hines Spring is 3 miles north of this spring, and has a continuous yield of about 4 sec-ft. Approximately, 1 sec-ft. finds its way into Owens River during the winter, but is lost during the remainder of the year. Campbell Spring is east of Owens River, 1 mile north of Aberdeen. It has a yield of about 0.5 sec-ft., and discharges directly into the river. Upper and Lower Seeley Springs are just above and just below Charlies Butte, and discharge directly into Owens River. The upper spring has a flow of 9.5 sec-ft., which is included in measurements of Owens River at the Butte. The lower one has a flow of 1.5 sec-ft.

Owens River.—Owens River flows lengthwise of the Independence Basin for 29 miles, although the actual length of its channel is possibly 20% greater, owing to its sinuosity. It is the drainage outlet for the waste surface water of the region, including the run-off from the valley floor, the yield of springs, and a small portion of the run-off from high mountain drainage areas. In order to account for all escaping surface waters, and determine the condition of the river channel with regard to seepage, observations of river discharge were made daily near the north and south boundaries of the region, and measurements of discharge into and diversion from the river channel were made between these two points. Complete data are available for 1909 and 1910. Analysis of these data shows that seepage losses occur during high-water, and seepage gains during low-water stages. The net result is a loss between Charlies Butte and Whitney Bridge, which can be accounted for by channel evaporation. The water plane of the valley on each side of the river lies between high- and low-water levels in the river. Hence, seepage gain and loss are the result of local ab-

sorption and drainage along the river channel, and have no relation to the general ground-water situation of the basin.

EVAPORATION AND TRANSPIRATION.

Evaporation from Water Surfaces.—Measurements of evaporation from free water surfaces were made under three conditions: from a pan floating in a body of water, from a pan placed in the soil, and from a deep tank placed in the soil. The first and second were designed to furnish data regarding evaporation from reservoir surfaces and from areas of shallow flood water, respectively. The third was desired for purposes of comparison with records of evaporation from soil. The pans, which were of the pattern used by the U. S. Reclamation Service, were 3 ft. square and 10 in. deep, and were of galvanized sheet-iron. Observations were made by replacing the quantity evaporated with a cup having a capacity equal to a depth of 0.01 in. in the pan. The initial height of the water surface was such that a pin, projecting from the center of the pan and remaining at a fixed height, 2 in. below the rim, was just submerged. The deep tank was circular, 3½ ft. in diameter and 4 ft. deep, and observations were made in a stilling well with a hook-gauge and vernier scale reading to 0.01 in. The records were all kept near Independence, and observations were made every second day in summer and every fourth day in winter.

The record for the pan in water (Table 6) is available from August 4th, 1908, to June 1st, 1911. The pan, Fig. 3, at first, was in Blackrock Slough, but was moved to its final location, in Owens River at Citrus Bridge, on May 7th, 1909. The pan was supported by a timber float, which protected it from splashing water. The depth of water beneath the pan varied from 1 to 5 ft., depending on the river stage. The river water had a moderate velocity, and varied in temperature from about 75° Fahr., in summer, to about 40° Fahr., in winter. The river banks averaged 4 ft. high above the water surface, and the pan was about 30 ft. from them. Rain gauge No. 18 was 100 ft. away, on the river bank, and was observed in connection with the evaporation record.

The record for the pan in soil is broken. It extends from August 1st, 1909, to November 30th, 1909, and from March 14th, 1910, to June 1st, 1911. The pan, Fig. 4, was in the valley floor at the soil evaporation experiment station, about 3 miles east of Independence.

It was set in a shallow excavation with soil banked up to about half the depth of the pan. Water temperatures range from 95° Fahr., in summer, to 32° Fahr., in winter. The surface temperature was about 1° warmer than that for the mixed contents of the pan. Table 7 summarizes the results by months for this pan, and, by comparing it month by month with the evaporation from the pan in water, an average excess of about 33% is observed. This is probably due to the higher temperature of the water in the pan in soil during the hours of sunlight.

TABLE 6.—DEPTH OF EVAPORATION, IN INCHES, FROM WATER SURFACE NEAR INDEPENDENCE (PAN IN WATER).

Month.	1908.		1909.		1910.		1911.		Average percentage of annual evaporation.
	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	
January	....	....	1.60	0.052	1.75	0.056	1.65	0.053	2
February	....	....	2.40	0.086	2.50	0.089	2.35	0.084	4
March	....	....	4.70	0.152	5.15	0.166	3.70	0.119	7
April	....	....	7.30	0.243	7.05	0.235	6.25	0.208	11
May	....	....	9.60	0.310	8.29	0.267	8.01	0.258	13
June	....	....	10.10	0.337	9.90	0.330	....	....	15
July	....	....	10.40	0.335	8.50	0.274	....	....	14
August	* 4.90	0.222	8.00	0.258	8.20	0.264	....	....	12
September	5.30	0.176	6.60	0.220	6.30	0.210	....	....	10
October	3.50	0.113	3.90	0.126	4.20	0.135	....	....	6
November	2.50	0.083	2.60	0.087	2.36	0.079	....	....	4
December	1.50	0.048	(1.85)	0.060	1.24	0.040	....	....	2
	....	....	69.05	0.189	65.44	0.179	....	....	100

* August 10th to 31st, inclusive.

The deep-tank record extends unbroken from April 16th, 1909, to Dec. 31st, 1911. The tank, Fig. 6, was at the soil evaporation experiment station, and was set in the soil with the upper rim flush with the surface. The water surface was not allowed to fall more than 4 in. below the rim. The temperature of the surface water varied from 80° Fahr., in the heat of summer to freezing in winter. Except during freezing weather, the average temperature of the contents of the tank was 5° less than that of the surface layer. The presence of the surrounding soil makes the range in temperature less than that for the shallow pan. The record, which is presented in Table 8, indicates an annual depth of evaporation practically equal to that from the pan in water at Citrus Bridge. The monthly distribution is more uniform,

TABLE 7.—DEPTH OF EVAPORATION, IN INCHES, FROM WATER SURFACE NEAR INDEPENDENCE (PAN IN SOIL).

Month.	1909. Total.	1909. Rate per 24 hours.	1909. Percentage of evaporation from pan in water.	1910. Total.	1910. Rate per 24 hours.	1910. Percentage of evaporation from pan in water.	1911. Total.	1911. Rate per 24 hours.	1911. Percentage of evaporation from pan in water.
January	......	......	......	......	......	......	2.25	0.073	138
February	......	......	......	......	......	......	2.25	0.080	95
March	......	......	......	* 4.25	0.236	......	4.80	0.155	130
April	......	......	......	9.50	0.316	135	8.12	0.271	130
May	......	......	......	10.61	0.342	128	10.25	0.330	128
June	......	......	......	11.95	0.398	121	......	......	......
July	......	......	......	12.55	0.405	148	......	......	......
August	10.70	0.345	134	11.80	0.381	144	......	......	......
September	8.50	0.283	129	8.80	0.293	140	......	......	......
October	5.80	0.187	149	5.60	0.180	133	......	......	......
November	3.80	0.127	146	2.85	0.095	121	......	......	......
December	......	......	......	1.60	0.052	129	......	......	......
	......	......	......	......	......	133	......	......	......

* March 14th to 31st, inclusive.

TABLE 8.—DEPTH OF EVAPORATION FROM WATER SURFACE NEAR INDEPENDENCE.

DEEP TANK IN SOIL.

Month.	1909. Total, in inches.	1909. Rate, in inches per 24 hours.	1909. Percentage of evaporation from pan in water.	1910. Total, in inches.	1910. Rate, in inches per 24 hours.	1910. Percentage of evaporation from pan in water.	1911. Total, in inches.	1911. Rate, in inches per 24 hours.	1911. Percentage of evaporation from pan in water.	Percentage of annual evaporation during each month.
Jan.	......	......	......	2.00	0.064	114	2.30	0.074	189	3
Feb.	......	......	......	2.90	0.104	116	2.55	0.091	108	4
Mar	......	......	......	5.60	0.180	109	3.95	0.127	107	7
Apr	2.90*	0.193	......	7.40	0.246	105	6.80	0.226	84	10
May	7.50	0.242	78	7.71	0.248	93	7.90	0.254	77	12
June	7.80	0.260	77	8.60	0.287	87	6.65	0.222	......	11
July	7.90	0.254	76	8.30	0.268	98	8.60	0.277	......	12
Aug	8.20	0.264	102	8.80	0.284	107	9.65	0.311	......	14
Sept	7.20	0.240	109	7.30	0.243	116	7.16	0.239	......	11
Oct.	5.00	0.161	128	5.15	0.166	123	4.90	0.158	......	7
Nov.	3.30	0.110	127	3.10	0.103	131	3.00	0.100	......	5
Dec.	(2.20)	0.071	119	2.15	0.069	173	(2.50)	0.081	......	4
Totals	......	......	......	69.01	0.188	106	65.96	0.180	....	100

* For period, April 16th to 30th, inclusive.

FIG. 6.—DEEP-WATER EVAPORATION TANK.

FIG. 7.—SOIL TANK NO. 3 IN OPERATION.

there being 70% of the total during the 6 summer months and a difference of 27 in. between summer and winter evaporation. The effect on evaporation of the modified temperature extremes of the soil is well shown by comparison with the record for the pan in water (Table 6). The temperature conditions for the deep tank agree closely with those of the surrounding soil.

Evaporation from Ground Surface.—Water in the surface layers of the ground is subject to evaporation, either directly from the soil or through vegetation by the process of transpiration. It is available for evaporation in Owens Valley under two conditions: temporarily, following a rainstorm or sudden thaw, and permanently, within areas where the average depth to ground-water does not exceed 8 ft. The total evaporation under the first condition is relatively unimportant, because of the infrequency of storms and the small quantity of precipitation, and no attempt was made to measure it. Under the second condition, however, evaporation losses are large, for, not only is soil capillarity able to draw gravity water to the surface, but roots of vegetation, such as wild grass, penetrate the soil to ground-water and become the channels by which a large quantity of moisture is conveyed into the atmosphere. Evaporation from bare soil combined with transpiration is, in fact, the most important element entering into computations relating to ground-water for this region. So few data are available on the subject that extended observations were undertaken.

Owens Valley is an ideal location for carrying on such experiments. In the first place, the source of water available for evaporation may be kept under the complete control of the observer as regards the quantity and rate of supply. Storms are rare, and the total precipitation is small, so that little uncertainty exists from this cause regarding the quantity of percolation from precipitation on the surface of a body of isolated soil. Second, the method by which the surface soils of the valley floor are kept moist can be reproduced artificially on a small scale with only a slight departure from natural conditions. The source of supply for soil moisture is a permanent ground-water surface from which water is drawn by capillary forces. This groundwater is replenished by percolation from the precipitation and surface water of the intermediate mountain and outwash slopes, which seeps laterally toward the valley floor and lies beneath it under hydrostatic pressure sufficient to maintain a permanent ground-water surface.

Similar pressure can be reproduced in the bottom layer of an isolated body of soil, and capillary forces can be depended on to raise moisture to the surface. Finally, the large annual depth of evaporation makes possible a more accurate determination of its quantity than in a less arid region. Experiments carried on under these conditions have been very satisfactory.

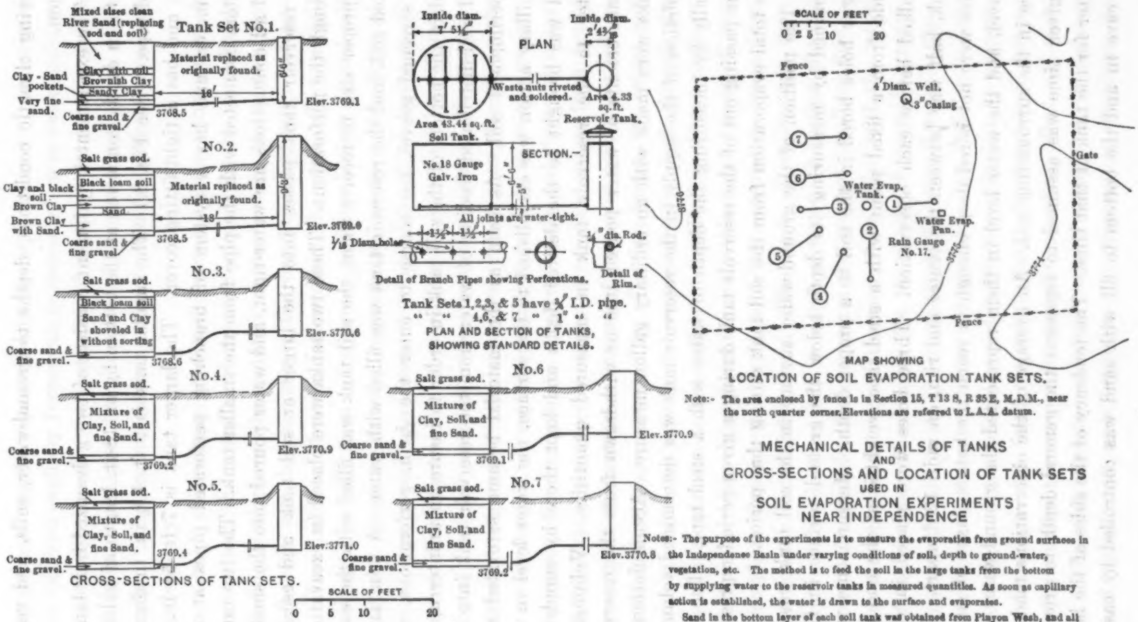
The rate of evaporation from soil depends on the temperature of the air and soil, the quantity of moisture already in the immediately surrounding atmosphere, the quantity of moisture in the surface layers of the soil, and the character of the vegetation and other soil covering. The first two of these factors have the same effect on soil evaporation as on that from a free water surface—higher air and soil temperatures result in increased evaporation, as does also dryer atmosphere or increased movement of wind. The third factor is directly proportional to the rate of evaporation, because the loss of moisture occurs from soil grains at or very near the surface. The quantity of moisture in the soil available for evaporation thus depends on the character of the soil, as regards capillarity and depth to the ground-water surface. For example, in a coarse, sandy soil, "gravity water" will be drawn to the surface through the capillary spaces from depths not exceeding 4 ft., and, in a fine sandy or clayey soil, water will be drawn from depths as great as 8 ft. The last factor, the extent and character of vegetation, affects the evaporation rate both through the activity of transpiration and the effect on capillarity. Plant roots are continually absorbing water from the soil; this water passes off into the atmosphere through the leaves, and the evaporation losses from soil are greatly increased thereby. The roots of native salt grass will penetrate to a depth of 8 ft. in search of water. A further effect of the growth of vegetation is to increase the vertical capillary flow of moisture through soil by way of the many tubes filled with the rotted fiber of dead roots. These tubes are the result of years of growth, and penetrate the soil in all directions above the ground-water surface.

The purpose of the experiments was to obtain data sufficiently complete to compute the total volume of water annually lost by evaporation and transpiration from the valley floor. This involved making observations under the various local conditions which affect soil evaporation. The plan was to reproduce natural conditions in isolated bodies of typical soil and determine the evaporation therefrom for

varying climatic conditions, depths to ground-water, soils, and vegetation.

The experimental equipment consists of two galvanized-iron tanks, 6½ ft. in depth, connected at the bottom by an 18-ft. length of galvanized pipe. (See Fig. 8.) The smaller tank is 2 ft. 4 $\frac{3}{16}$ in. in diameter, and has a tight-fitting cover. The larger tank is 7 ft. 5¼ in. in diameter, and has a system of branching perforated pipes at the bottom connected with the pipe from the smaller tank. The two tanks and all connections are water-tight, and water poured into the smaller or reservoir tank passes into the larger or soil tank and escapes through the perforations. These two tanks were placed in excavations of proper size to receive them, the soil tank was filled with the excavated soil, and the reservoir tank was filled with water. A 6-in. layer of screened gravel, too coarse to enter the $\frac{1}{16}$-in. perforations, was laid in the bottom of the soil tank in order to insure an uninterrupted and well-distributed feeding of water from the reservoir tank into the superimposed soil. As soon as the material became saturated and capillary action was established to the surface, the water level in the soil was brought to the desired depth and kept there by supplying water to the reservoir tank in measured quantities. Volumetric measurements of water poured into or withdrawn from the reservoir tanks were made with an ordinary gallon measure. Accumulation or depletion of the supply in the reservoir tank was determined volumetrically by measuring the depth of water with a steel tape. The volume passing out of the reservoir tank during a given period represents the total evaporation from the soil tank during that period.

The position of the ground-water surface in the soil tank was determined by measuring its depth below the ground surface in 2-in. augur holes bored in the soil to a proper depth. Measurements were made from a fixed point with a steel tape weighted at the end and chalked before each observation. Three holes were placed in each tank, half way between the center and rim, on radii 120° apart. The holes were not bored deep enough to reach the bottom layer of coarse gravel, and the water level in them represented the ground-water surface in the surrounding soil. An average of the observations made at a given time was assumed to represent the general depth to ground-water for the tank at that time. The tendency of the sides of the holes to cave in and the bottom to fill with sand was controlled by casing



MAP SHOWING
LOCATION OF SOIL EVAPORATION TANK SETS.

Note:- The area enclosed by fence is in Section 15, T 13 S, R 35 E, M.D.M., near the north quarter corner. Elevations are referred to L.A.A. datum.

MECHANICAL DETAILS OF TANKS
AND
CROSS-SECTIONS AND LOCATION OF TANK SETS
USED IN
SOIL EVAPORATION EXPERIMENTS
NEAR INDEPENDENCE

Notes:- The purpose of the experiments is to measure the evaporation from ground surfaces in the Independence Basin under varying conditions of soil, depth to ground-water, vegetation, etc. The method is to feed the soil in the large tanks from the bottom by supplying water to the reservoir tanks in measured quantities. As soon as capillary action is established, the water is drawn to the surface and evaporates.

Sand in the bottom layer of each soil tank was obtained from Piayon Wash, and all sizes less than 1/16" were screened out.

FIG. 8.

them with 2-in. galvanized sheet-iron pipe generously perforated with $\frac{1}{16}$-in. holes. These pipes were driven so that the top was just flush with the ground surface, and they were closed at the top with wooden plugs. In some of the tanks it was found impossible to bring the ground-water surface to the desired level with the available hydrostatic pressure from the reservoir tanks, and 2-in. holes were bored between the observation holes to the saturated gravel layer. Water usually rose in these holes to the same height as in the reservoir tank, and, by seeping laterally into the soil, built up the ground-water surface. It was found difficult to keep these holes open to the gravel, however, and the water level in most of them eventually represented the ground-water surface.

Three tank sets were installed in the open valley floor east of Independence in February, 1909. The surface of Soil Tank No. 1 was bare sand; Nos. 2 and 3 (Fig. 7) were laid with salt-grass sod. The initial plan formulated for Tank Sets Nos. 1 and 3 was to hold the ground-water level at various depths below the ground surface for periods of a few weeks during the summer while the climatic conditions were constant, in order to obtain, in a short time and with few tanks, trustworthy results of a general nature. The movement of the water surface from one level to another consumed so much time, however, that winter approached before the experiments on the lower levels were reached, and furthermore, there was no accurate method of determining the volume of evaporated water represented by the differences in depth. The experience of the first year's work with these tanks showed the necessity of maintaining a fixed ground-water level during a complete cycle of climatic changes. In Soil Tank No. 2 it was at first proposed to hold the ground-water level at or near the ground surface, but so great was the rate of summer evaporation that this plan was found to be impracticable with the equipment available. To remedy the defect, the hydrostatic pressure from the reservoir tank was increased by soldering to it a 3-ft. extension, but this was not used until late in the season. This experience suggested the desirability of placing the reservoir tanks above the soil tanks and of increasing the size of the feed pipe.

As a result of these preliminary observations, four additional tank sets were installed in January, 1910. The reservoir tank outlets were placed about 1.7 ft. above the soil tank inlets, and 1-in. pipe was used

throughout. The new soil tanks were laid with salt-grass sod, which took root and grew in every tank. The general plan of operation for Tank Sets Nos. 2 to 7 was to supply the reservoir tanks with water in quantities such that the depths to ground-water in the soil tanks were, respectively, 5 ft., 4.5 ft., 4 ft., 3 ft., 2 ft., and 1 ft. Observations were carried on continuously on the six tanks during the two years, 1910

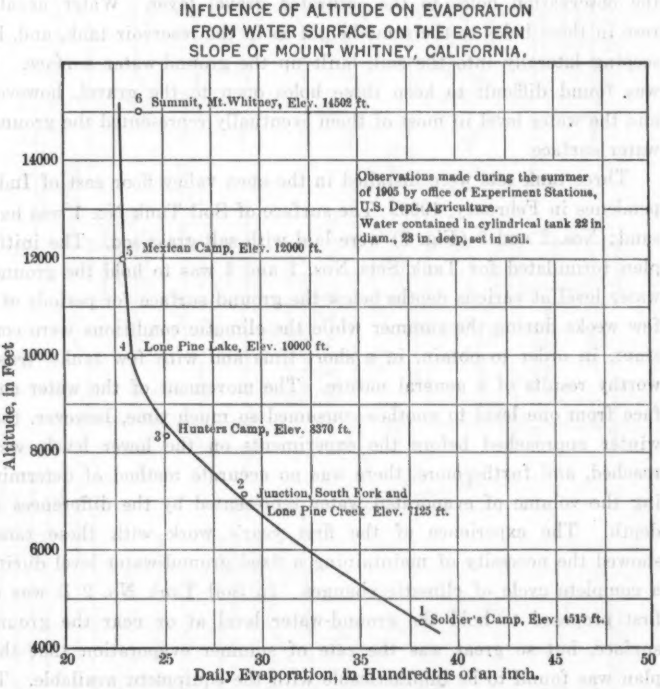


FIG. 9.

and 1911. The operation and results were quite satisfactory, with the exception that in Tank Set No. 7 the pressure from the reservoir tank was not sufficient between April and September to hold the water level at the 1-ft. depth.

An important feature of reproducing natural conditions for combined soil evaporation and transpiration is to obtain a fully developed root system reaching down to the ground-water surface. At best, this

requires more than a year, particularly for the greater depths. In order to stimulate the growth as much as possible, the water level in all soil tanks was brought up to about 1 ft. below the surface as soon after installation as possible. This was accomplished by pouring water into the observation holes until the soil was completely saturated to the level desired and the surface showed moisture. Then no water was added to the reservoir tanks until the ground-water level had receded by evaporation and transpiration to the desired level. The grass roots were thus given a good initial irrigation and an opportunity to follow the water down. Active growth occurred in Tanks Nos. 4 to 7 during the first year, and continued with greater vigor during the second year. In Tank No. 3 a less active growth occurred the first year, but the results were more satisfactory during the second year. There was practically no growth in Tank No. 2 during the first year, although the grass did not die. During the second year the grass showed more signs of life, but did not grow as actively as in Tank No. 3.

The details of the observations on soil evaporation for 1910 and 1911 for Tank Set No. 5 are shown graphically on Plate IV. The supply of water available to the soil from the reservoir tank is the element under complete control of the observer, and at the top of the diagram are statements of the purpose governing additions to or withdrawals from this supply during various periods of time. Below this is platted a broken line representing the fluctuation of water surface in the reservoir tank, the vertical portions indicating additions to or withdrawals from the reservoir supply made by the observer, and the inclined portions indicating the soil-tank draft. There is also platted a mass-curve showing the aggregate volume of water supplied to the reservoir tank, which appears as a series of vertical and horizontal lines. At the bottom of the diagram is platted an undulating line representing the fluctuation of ground-water surface in the soil tank, each depth being obtained by averaging the depths recorded in the observation holes.

The small part that precipitation plays in ground-water fluctuations in Owens Valley is shown by this diagram. The average annual precipitation at the experiment station is about 4.38 in., the season, 1909-10, being normal and 1910-11 well above normal. At the bottom of the diagram is noted the date and quantity of precipitation for each storm. It is seen that, even in a wet season, percolating water does not penetrate to depths exceeding 2.5 ft., unless more than 1 in. falls

within a short period on moist soil. Even then it does not appear to reach depths greater than 4 ft. The problem of percolation from rainfall, therefore, is practically eliminated from the experiments. When rising ground-water was noted in a soil tank after precipitation, the volume of percolating water was estimated from the observed rise and included in the mass-curve, as noted on the diagrams.

The quantity of water evaporated from the soil surface of any tank during a given period can be computed accurately from the diagrams, when the depth to ground-water at the beginning and end of the period is the same, by noting from the mass-curve the quantity supplied to the reservoir tank during the period and the accumulation or depletion in the reservoir tank. The sum of these quantities, with their proper algebraic sign, gives the loss by evaporation. For differing depths to ground-water, however, the computations are only approximate, because the proportion of empty space in the soil layer and the quantity of moisture it contained initially are both unknown. A monthly summary of results for Tank Sets Nos. 2 to 7 for 1911 is presented in Tables 9 to 14. The annual depth of evaporation from the several soil tanks exhibited a consistent decrease with increase of depth to ground-water, and varied from 48.8 in. for No. 7 to 13.43

TABLE 9.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 2.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan.	6	1.10	1.15	+ 2	4	0.15	0.005	4.98
Feb.	0	1.15	1.11	− 1	1	0.04	0.001	4.95
Mar.	3	1.11	1.14	+ 1	2	0.07	0.002	4.94
Apr.	10	1.14	1.18	+ 1	9	0.33	0.011	4.94
May	30	1.18	1.19	0	30	1.11	0.036	4.98
June	66	1.19	1.39	+ 6	60	2.22	0.074	5.03
July	71	1.39	1.22	− 5	76	2.81	0.091	5.00
Aug.	105	1.22	1.78	+ 18	87	3.22	0.104	4.95
Sept.	52	1.78	1.52	− 8	60	2.22	0.074	4.80
Oct.	18	1.52	1.37	− 5	23	0.85	0.028	4.80
Nov.	1	1.37	1.18	− 6	7	0.26	0.009	4.85
Dec.	1	1.18	(1.10)	− 3	4	0.15	0.005	4.98
Year.	363	1.10	1.10	0	363	13.43	0.037	4.94

PLATE IV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1315.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.

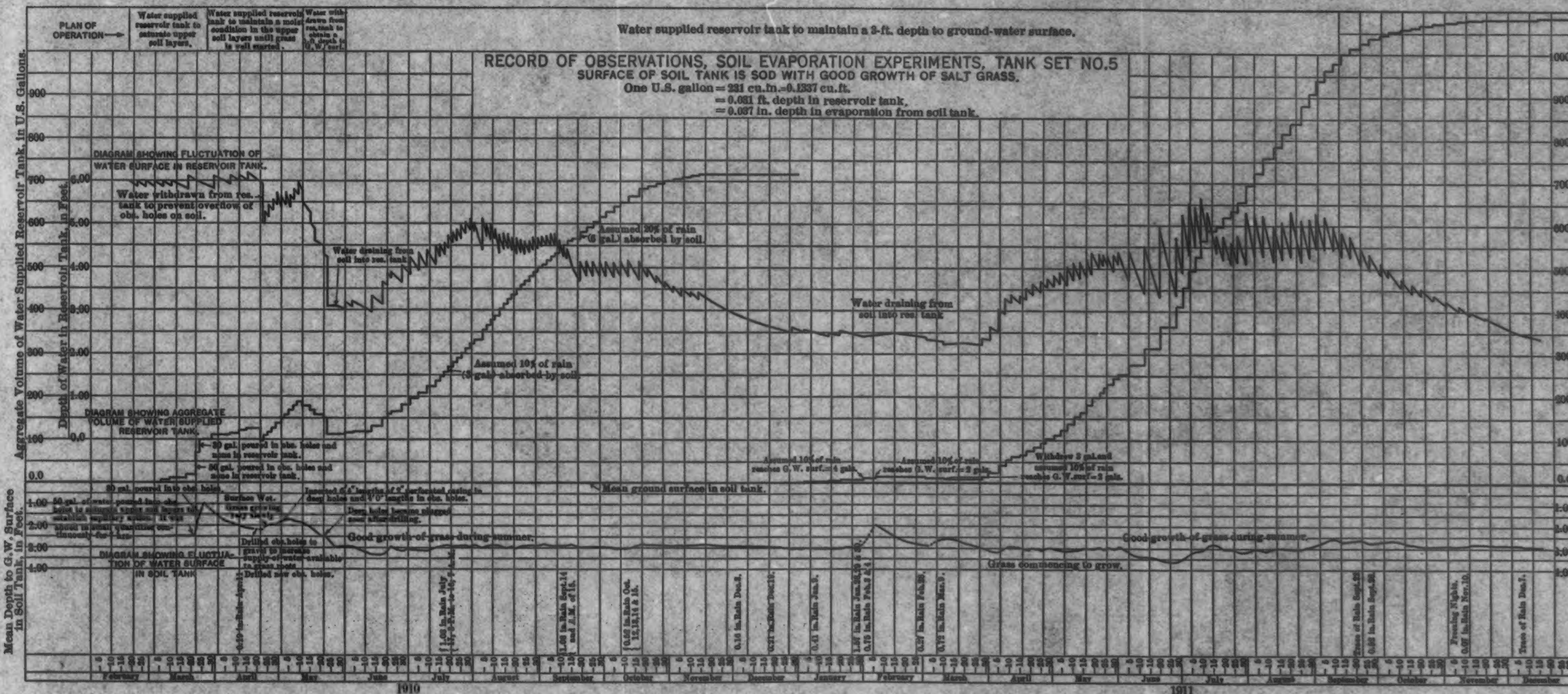


TABLE 10.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 3.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet. Beginning of month.	End of month.	Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated. Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	Average depth to ground-water surface in soil tank, in feet.
Jan.......	9	0.14	0.10	— 1	10	0.37	0.012	4.58
Feb.......	18	1.10	0.23	+ 4	14	0.52	0.019	4.59
Mar.......	12	0.23	0.20	— 1	13	0.48	0.015	4.48
Apr.......	40	0.20	0.54	+ 11	29	1.07	0.036	4.50
May.......	62	0.54	0.60	+ 2	60	2.22	0.072	4.49
June.......	103	0.60	0.96	+ 12	91	3.37	0.112	4.63
July.......	160	0.96	0.67	— 9	169	6.25	0.202	4.51
Aug.......	285	0.67	2.00	+ 43	242	8.96	0.289	4.52
Sept.......	135	2.00	1.13	— 28	163	6.03	0.201	4.22
Oct.......	27	1.13	0.54	— 19	46	1.70	0.055	4.21
Nov.......	4	0.54	0.16	— 12	16	0.59	0.020	4.33
Dec.......	3	0.16	(0.10)	— 2	5	0.18	0.006	4.56
Year.....	858	0.14	0.10	0	858	31.74	0.087	4.46

TABLE 11.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 4.

Month.	Volume of water supplied to reservoir tank in gallons.	Depth of Water in Reservoir Tank, in Feet. Beginning of month.	End of month.	Accumulation or depletion of water in reservoir tank,in gallons.	Volume of Water Evaporated. Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	Average depth to ground-water surface in soil tank, in feet.
Jan.......	7	1.53	1.49	— 1	8	0.30	0.010	3.97
Feb.......	— 8	1.49	1.18	— 10	2	0.07	0.002	3.36
Mar.......	3	1.18	1.19	0	3	0.11	0.004	3.74
Apr.......	62	1.19	2.15	+ 31	31	1.15	0.038	4.00
May.......	96	2.15	2.57	+ 14	82	3.04	0.098	4.06
June.......	121	2.57	2.63	+ 2	119	4.40	0.147	(3.34)
July.......	114	2.63	1.97	— 21	135	5.00	0.161	3.44
Aug.......	141	1.97	2.47	+ 16	125	4.63	0.149	3.92
Sept.......	65	2.47	1.90	— 18	83	3.07	0.102	3.85
Oct.......	30	1.90	1.53	— 12	42	1.55	0.050	3.93
Nov.......	9	1.53	1.09	— 14	23	0.85	0.028	3.97
Dec.......	6	1.09	(0.91)	— 6	12	0.44	0.015	4.10
Year.....	646	1.53	0.91	— 19	665	24.61	0.067	3.81

TABLE 12.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 5.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of water in reservoir tank, in feet. Beginning of month.	End of month.	Accumulation or depletion of water in reservoir tank, in gallons.	Volume of water evaporated. Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	Average depth to ground-water surface in soil tank, in feet.
Jan	8	2.55	2.43	− 4	12	0.44	0.014	3.01
Feb	6	2.43	2.42	0	6	0.22	0.008	2.47
Mar	3	2.42	2.36	− 2	5	0.18	0.006	2.73
Apr	88	2.36	3.65	+ 42	46	1.70	0.057	2.99
May	141	3.65	4.09	+ 14	127	4.70	0.151	3.01
June	166	4.09	4.19	+ 3	163	6.03	0.201	3.40
July	244	4.19	4.14	− 2	246	9.11	0.294	3.06
Aug	255	4.14	4.92	+ 23	232	8.58	0.280	2.99
Sept	127	4.92	4.07	− 25	152	5.62	0.188	2.69
Oct	36	4.07	3.24	− 27	63	2.33	0.075	2.77
Nov	9	3.24	3.67	− 18	27	1.00	0.033	2.83
Dec	1	2.67	(2.45)	− 7	8	0.30	0.010	2.90
Year	1 084	2.55	2.45	− 3	1 087	40.21	0.110	2.90

TABLE 13.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 6.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of water in reservoir tank, in feet. Beginning of month.	End of month.	Accumulation or depletion of water in reservoir tank, in gallons.	Volume of water evaporated. Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	Average depth to ground-water surface in soil tank, in feet.
Jan	12	3.03	3.19	+ 5	7	0.26	0.008	1.92
Feb	9	3.19	3.17	− 1	10	0.37	0.013	1.31
Mar	24	3.17	3.32	+ 5	19	0.70	0.023	1.70
Apr	94	3.32	3.53	+ 7	87	3.22	0.107	2.00
May	163	3.53	3.54	0	163	6.03	0.195	1.99
June	182	3.54	3.75	+ 7	175	6.48	0.216	2.30
July	213	3.75	3.31	−14	227	8.40	0.271	2.02
Aug	314	3.31	4.25	+30	284	10.50	0.339	2.01
Sept	143	4.25	3.89	−12	155	5.74	0.192	1.55
Oct	38	3.89	3.36	−17	55	2.04	0.065	1.65
Nov	8	3.36	2.98	−12	20	0.74	0.025	1.84
Dec	6	2.98	(2.83)	− 5	11	0.41	0.013	2.04
Year	1 206	3.03	2.83	− 7	1 213	44.89	0.122	1.86

TABLE 14.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

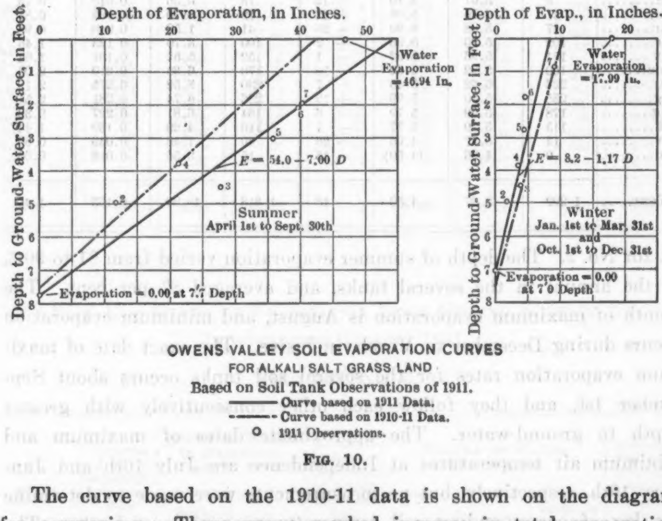
TANK SET NO. 7.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet. Beginning of month.	End of month.	Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated. Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	Average depth to ground-water surface in soil tank, in feet.
Jan........	8	5.07	4.70	− 12	15	0.56	0.018	0.73
Feb........	31	4.70	5.08	+ 12	19	0.70	0.025	0.81
Mar........	67	5.08	5.90	+ 26	41	1.52	0.049	0.98
Apr........	102	5.90	5.96	+ 2	100	3.70	0.123	1.46
May	151	5.96	5.92	− 1	152	5.62	0.181	1.64
June.......	160	5.92	5.62	− 10	170	6.30	0.210	2.06
July.......	223	5.62	5.40	− 7	230	8.52	0.275	2.19
Aug........	255	5.40	5.99	+ 19	236	8.74	0.281	2.51
Sept.......	178	5.99	5.79	− 6	184	6.81	0.227	2.39
Oct........	115	5.79	5.77	− 1	116	4.29	0.139	1.44
Nov........	14	5.77	4.96	− 26	40	1.48	0.049	0.60
Dec........	0	4.96	(4.50)	− 15	15	0.56	0.018	0.60
Year.....	1 299	5.07	4.50	− 18	1 318	48.80	0.133	1.45

in. for No. 2. The depth of summer evaporation varied from 81 to 90% of the annual in the several tanks, and averaged 87 per cent. The month of maximum evaporation is August, and minimum evaporation occurs during December to March, inclusive. The exact date of maximum evaporation rates for the several soil tanks occurs about September 1st, and they follow each other consecutively with greater depth to ground-water. The approximate dates of maximum and minimum air temperatures at Independence are July 10th and January 10th, respectively, but no measurements were made to determine the lag of corresponding soil temperatures at various depths. The extremes of evaporation from water surfaces agree in time of occurrence with maximum and minimum air temperatures, however, and the observed lag in soil evaporation is in general consistent with the observed lag in soil temperatures in other localities. Hence it is reasonable to conclude that extremes in the rate of soil evaporation and soil temperature are concurrent at a given depth.

A graphic study of the data in Tables 9 and 14 for the periods, April 1st to September 30th, and October 1st to March 31st, which are, respectively, periods of increasing and decreasing evaporation rate, is presented in Fig. 10. There appears to be, during each period,

a straight-line relation between total evaporation and depth to ground-water. The limiting depth is 7.7 ft., and the total evaporation when water and ground surface coincide, 54.0 and 8.2 in., respectively. The total depth of evaporation, in inches, being represented by E, and the depth to ground-water, in feet, by D, the equations representing variation in evaporation with depth to ground-water are $E = 54.0 - 7.00\,D$ and $E = 8.2 - 1.17\,D$. It will be noted that the combined soil evaporation and transpiration during the summer exceeded the water evaporation from the tanks in the soil by 15 per cent.



OWENS VALLEY SOIL EVAPORATION CURVES
FOR ALKALI SALT GRASS LAND
Based on Soil Tank Observations of 1911.
——— Curve based on 1911 Data.
-·-·-·- Curve based on 1910-11 Data.
o 1911 Observations.

Fig. 10.

The curve based on the 1910-11 data is shown on the diagram for comparison. There was a marked increase in total evaporation during 1911, as compared with the season, 1910-11, which was due to the more complete development of the root systems. Broadly speaking, the results for 1911 showed an increase of 17% in the volume of water consumed during the two periods. A continuation of observations for another year would show still further increase for those tanks in which the grass roots had not reached the water plane. Observations of depth to water in test holes in the transition zone between meadow and desert land indicate that soil evaporation ceases for depths exceeding 8 ft. The effect of increased evaporation for the tanks with greatest depth to ground-water would be to drop the lower end of the

curve to some point below 7.7 ft. The true curve, therefore, is probably steeper than that for 1911, crossing the X-axis at about the same point, but the Y-axis at about 8 ft. instead of 7.7 ft. However, in the practical use of the curve, the departure of the lower end from the true position does not affect materially the computations of the total volume of water evaporating from a given area, as the proportion of such volume originating in areas of relatively deep ground-water is small.

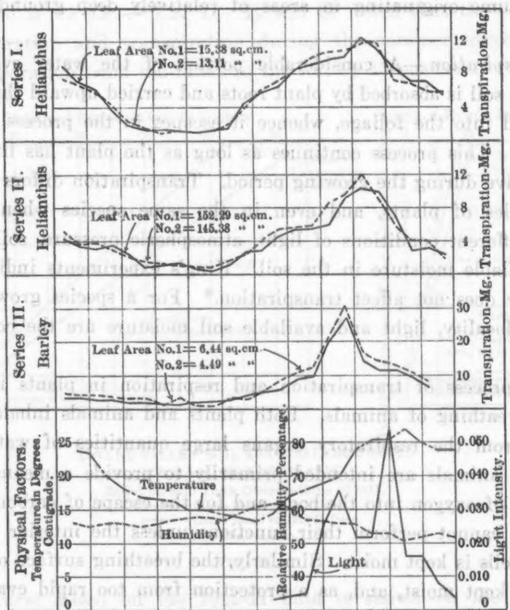
Transpiration.—A considerable portion of the water evaporating from the soil is absorbed by plant roots and carried upward through the stem and into the foliage, whence it escapes in the process of transpiration. This process continues as long as the plant has life, but is most active during the growing period. Transpiration differs in different species of plants, and even in the same species when existing under different conditions of light, atmospheric pressure, soil texture, and available moisture in the soil. King's experiments indicate that humidity does not affect transpiration.* For a species growing in a definite locality, light and available soil moisture are the controlling factors.

The process of transpiration and respiration in plants is similar to the breathing of animals. Both plants and animals inhale air and exhale from the respiratory organs large quantities of water. The lungs of animals are intended primarily to provide a means for the entrance of oxygen into the body and for the escape of carbon dioxide, but they cannot perform their functions unless the interior lining of the air cells is kept moist. Similarly, the breathing surface of a plant must be kept moist, and, as a protection from too rapid evaporation, this surface is within the plant structure, principally in the foliage. Plant leaves are enclosed in a relatively impervious skin or epidermis in which are small breathing pores or stomata which open or close automatically, depending on the needs of the plant for a greater or less quantity of air. When exposed to light, the food-manufacturing processes of a green plant are stimulated, and require a continually changing volume of air in contact with the breathing surface. The stomata open proportionally to the light intensity. Should the water supply in contact with the roots be insufficient, the breathing surface may become dry, and when that happens the stomata close automatically

* "Irrigation and Drainage," by F. H. King. New York, 1899.

until the proper quantity of air is admitted for the plant to do its work under the new conditions. The stomata, therefore, control the quantity and rate of loss of water from plants by transpiration.

There is a marked diurnal periodicity in the rate of transpiration, which investigators are led to believe is largely the result of the varying intensity of light. This periodicity is well illustrated by ob-



RELATION OF TRANSPIRATION TO LIGHT INTENSITY DURING A 24-HOUR DAY.

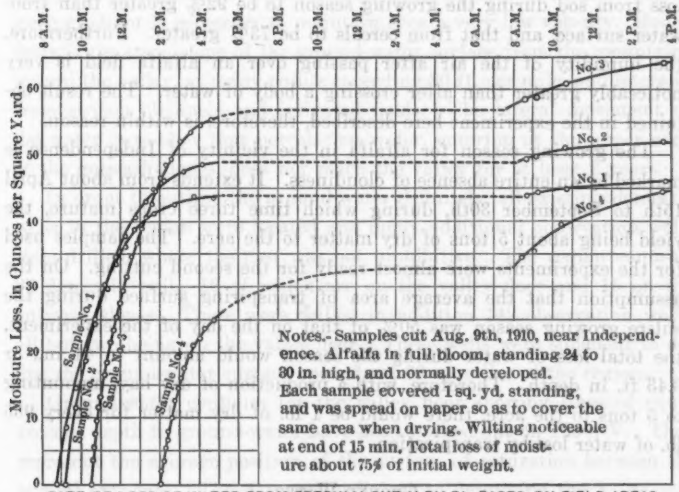
FIG. 11.

servations made under the direction of Mr. Frederick E. Clements, State Botanist of Minnesota, and reproduced in Fig. 11.* Measurements of transpiration were made hourly from 6 P. M. on February 16th to 6 P. M. on February 17th, and the physical factors were observed between these hours. The day was cloudy throughout, so that

* "Influence of Physical Factors on Transpiration", by A. W. Sampson and L. M. Allen, Minnesota Bot. Studies, Pt. I, Vol. 4, 1909, p. 42.

the variation in temperature and humidity was slight. The diagrams show very strikingly the response of transpiration to changes in intensity of light.

No measurements of transpiration are available for conditions similar as regards altitude and aridity to those in Owens Valley. It is unnecessary in this study to know separately the transpiration from wild grasses and the evaporation from bare soil, because the area of the latter is relatively small. The experiments on soil evaporation, therefore, were planned to give the combined loss from these two causes. It is desirable, however, to know the quantity of transpiration from field crops, in order to aid in computing the quantity of percolation from irrigation. Observations for such crops were confined to alfalfa.



RATE OF LOSS OF WATER FROM FRESHLY CUT ALFALFA, BASED ON FIELD AREAS.

FIG. 12.

The method of measurement was based on the assumption that the rate of loss of water from freshly cut plants would correspond closely to the rate before cutting. The plants were cut rapidly from a measured area, weighed, and spread out on paper to cover the same area as before cutting. At short intervals they were reweighed until there was no further appreciable loss. No noticeable wilting occurred during the first 15 min. and the rate of loss during this period was used as

a basis for calculations. The results of the measurements are shown graphically in Fig. 12.

The rapid decrease in the rate of loss is very noticeable. Inspection of Fig. 11 will show that the rates of transpiration at 8.45, 9.15, and 10.30 A. M., and 2.02 P. M., expressed as percentages of the average rate for 24 hours, are, respectively, 128, 141, 177, and 197, an average of 161 per cent. If a similar relation is assumed, the average loss in a 24-hour day from the four alfalfa samples would be 366 oz. per sq. yd. of field area, or 0.49 in. in depth. This figure appears to be rather large, at first glance, for the rate of evaporation for that day from the pan in Owens River was 0.30 in., and that from the shallow pan in the soil was 0.38 in. The results obtained by German investigators indicate the loss from sod during the growing season to be 92% greater than from water surface, and that from cereals to be 73% greater. Furthermore, the humidity of the air after passing over an alfalfa field is very noticeably greater than after crossing a body of water. The result obtained in the experiment here described, therefore, is within reason.

The growing season for alfalfa in the vicinity of Independence is marked by an entire absence of cloudiness. It extends from about April 15th to September 30th, during which time three crops mature, the yield being about 5 tons of dry matter to the acre. The samples used for the experiments were almost ready for the second cutting. On the assumption that the average area of transpiring surface during the entire growing season was 50% of that on the day of the experiment, the total loss of water during the season would amount to 41 in., or 3.43 ft. in depth. Therefore, with a production of dry hay, amounting to 5 tons to the acre, there would be 1 lb. of dry matter for every 935 lb. of water lost by transpiration.

GROUND-WATER.

Form of the Ground-Water Surface.—The general form of the ground-water surface corresponds with that of the surface of the valley fill, although the slopes are less steep and the irregularities are not so pronounced. In the valley floor the depth to ground-water is only a few feet. It becomes progressively greater toward the mountains, and probably lies 200 or 300 ft. beneath the outwash slope at about the 5 000-ft. contour. Superimposed on the general ground-water surface are sharp "ridges" beneath stream channels and "mounds" under irrigated

fields. The surface of the water in the underground reservoir, therefore, is not a level plain, but has a varied topography.

There are two reasons for this condition: the action of gravity tending to equalize inequalities in the ground-water surface, and the resistance which the ground offers to the lateral motion of water through its interstices. Percolating waters enter the valley fill from the upper edge of the outwash slope, from stream channels crossing the outwash slope, and from irrigated fields. The valley floor is the lowest portion of the valley fill and also the ground-water outlet. The force of gravity, therefore, tends to draw percolating water which has reached the surface saturation to the level of the valley floor. This can occur only by a lateral movement of water from the outwash slope toward the valley floor, but the resistance of the porous material is so great that a steep gradient is necessary to maintain even a very low velocity. Hence there is the steep slope of the ground-water surface from the mountains toward the valley, at many points exceeding 80 ft. per mile, and laterally from stream channels and irrigated fields. The lateral movement of the water is so slow that percolating water, entering at the upper edge of the outwash slopes, does not reach the valley for at least 2 years.

In order to outline the ground-water definitely, all existing domestic wells in the region were located and many additional observation wells were drilled, where the cost was not prohibitive. The region contains 27 domestic wells, 12 of which are on the valley floor and 15 on the outwash slopes. There were drilled in addition 142 observation wells, all but two being on the valley floor. These wells were sufficient to define the ground-water surface over about 60 sq. miles of the region.

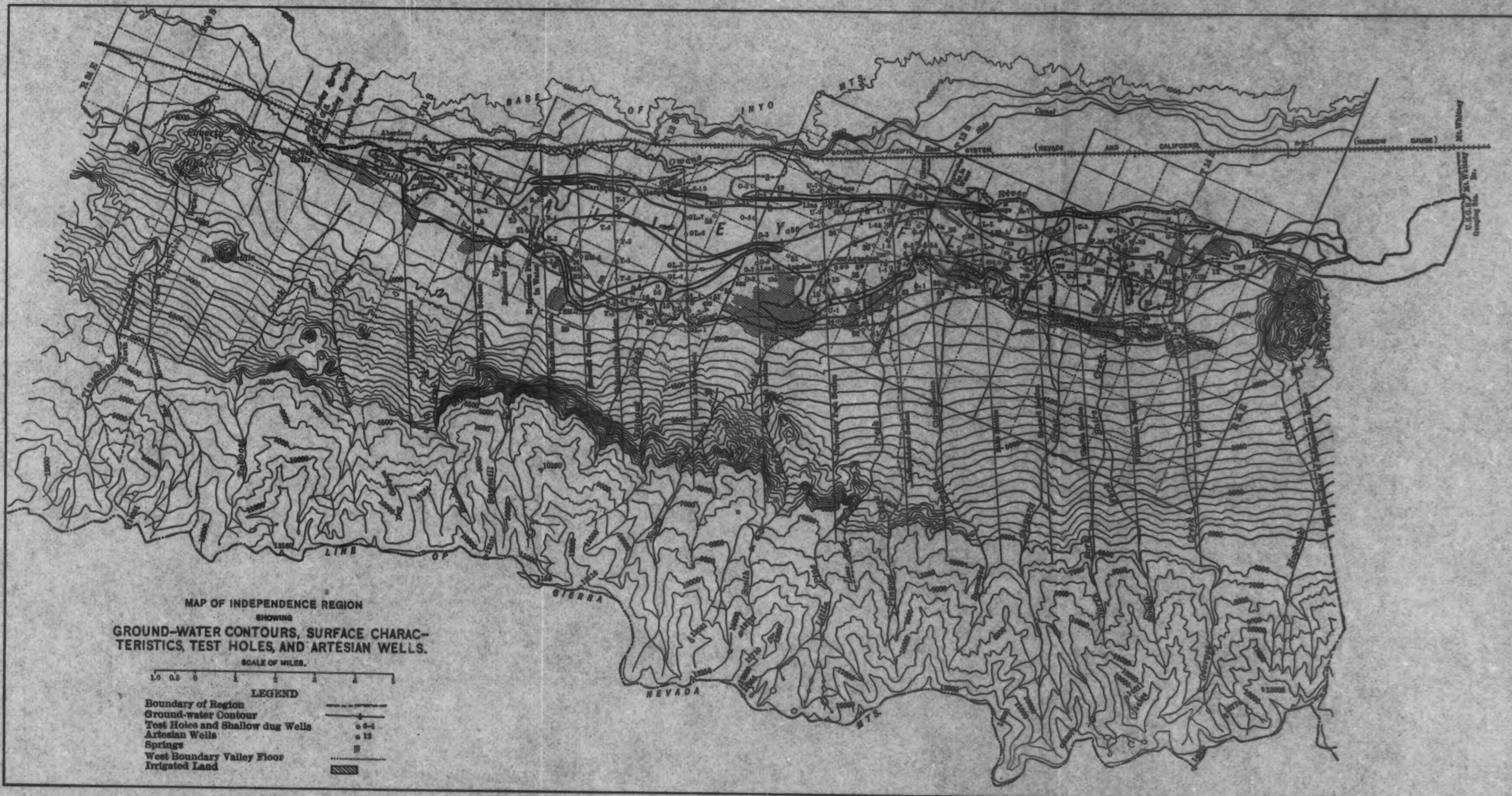
Ground-water contours for the valley floor showing lines of equal average depth to ground-water have been worked out on Plate V. They represent the average position of the surface of saturation between the annual extremes. The data are sufficient to determine the 3-, 4-, and 8-ft. contours with reasonable accuracy. The sudden approach of ground-water toward the surface at the upper edge of the grass land is shown, and also the general proximity of ground-water to the surface throughout the valley floor. The total area between the westerly 8-ft. contour and Owens River is 67 sq. miles. The average depth to ground-water is between 4 and 8 ft. over 40% of this area, and between 3 and 4 ft. over 28 per cent. It exceeds 8 ft. over 14% of the area, and is 3 ft. or less over 18 per cent. The area of the valley floor is 66.4 sq. miles

(Table 1), and its west boundary practically coincides with the 8-ft. contour.

There is a very striking relation between vegetation and depth to ground-water. On the outwash slopes the vegetation consists of various stunted desert shrubs. In approaching the valley floor at about the 20-ft. contour, sagebrush begins to predominate, and has a luxuriant growth as far east as the 12-ft. contour, where it is replaced by greasewood, rabbit brush, and coarse bunch grass. In the vicinity of the 8-ft. contour, salt grass begins to appear, and farther east, near and within the area inclosed by the 4-ft. contour, it grows luxuriantly. Within the 3-ft. contour, fresh-water grasses thrive where there is sufficient surface water to leach out and carry away most of the alkali, but the salt grass grows well, even where the soil is alkaline. In various portions of the valley floor rabbit brush and greasewood are found where the average depth to ground-water is 4 ft. or more, but grass predominates east of the 8-ft. contour. In areas where the alkali is excessive there is practically no vegetation. In general, grass does not grow where the depth to ground-water exceeds 8 ft., so the 8-ft. contour tends to coincide with the boundaries between meadow and desert lands.

Fluctuation of the Ground-Water Surface.—The surface of the ground-water is continually fluctuating. Both the extent and character of this fluctuation vary widely in different localities and at different times, depending on the proximity to ground-water sources or outlets and the relative rates of ground-water accretion and depletion. Three pronounced types are to be observed in the Independence Basin: (1) broad irregular fluctuations of varying amplitude in the outwash-slope area; (2) slightly irregular periodic fluctuation with wide fixed limits in and near irrigated areas; and (3) a regular periodic fluctuation with comparatively narrow and fixed limits in the valley floor. Special characteristics are also exhibited by wells within certain limited areas, as the result of local ground-water conditions.

These fluctuations were determined and studied from well observations made by the methods already described. Readings obtained at intervals of from 2 to 4 weeks were sufficient to establish accurately the position of the ground-water surface, as the fluctuations are characterized by great regularity. Most of the wells were observed from August 15th, 1908, to November 15th, 1909, and on 26 of the most



MAP OF INDEPENDENCE REGION
SHOWING
GROUND-WATER CONTOURS, SURFACE CHARACTERISTICS, TEST HOLES, AND ARTESIAN WELLS.

typical wells observations were continued to May 1st, 1911. The fluctuation of the surface of the lake south of Citrus Bridge was observed from August 15th, 1908, to November 15th, 1909, and of Goose Lake from August 15th, 1908, to May 1st, 1911.

The type of fluctuation peculiar to wells on the outwash slope is shown on Plate VI by Wells Nos. 31, 64, 25, 26, and 59, and Citrus No. 1. Water stands 10 ft. or more below the surface in all these wells, the vegetation of the surrounding area is limited to desert shrubs, and there are no alkali deposits on the surface. With knowledge of the sources and movements of ground-water beneath the outwash slopes, the assigned cause for this type would be annual variation in the quantity of water supplied by percolation from precipitation on the intermediate and outwash slopes and from stream channels. This is confirmed by the observations. For example, Well No. 31, which is 7 miles from the base of the Sierra and 500 ft. south of the old channel of Pinyon Creek, exhibits a persistent downward tendency which was partly checked during the summer of 1909 and 1910. The maximum effect of the very wet years, 1906 and 1907, evidently reached this well in 1908 and early in 1909. During the following years the water had a tendency to return to its normal level. This was twice opposed by percolation from the channel of Pinyon Creek, which carried flood-water during a few weeks in June and July, 1909, and for a very short period in 1910. Citrus Well No. 1, which is about ¾ mile south of Well No. 31, has similar fluctuations, but in it the maximum effect of seepage from Pinyon Creek is registered 6 weeks later in 1909, and in 1910 is much smaller in quantity. Well No. 64, situated similarly with respect to the mountains, but north of Little Pine Creek, has the same downward tendency, which is checked temporarily during the summer by irrigation in a near-by alfalfa field and a small garden at the well. Well No. 59, which is 2 miles from the base of the Sierra and ½ mile south of Sawmill Creek, had an upward tendency during 1909, due to the percolation from precipitation of the wet winter, 1908-09. In 1910 the water level fell in response to the normal winter of 1909-10. Seepage from Sawmill Creek does not affect this well appreciably. Wells Nos. 25 and 26 exhibit the general tendencies of Well No. 59, but they are in the transition zone between the outwash slope and the valley floor, where there is a periodic back-water effect from the annual rise of ground-water in the grass land.

Wells Nos. 61 and 62 illustrate the type of fluctuation characteristic of the irrigated areas of the region. They are in irrigated gardens in the Town of Independence. The form of the curve is periodic, with sharp crests and troughs, the former in July, the latter in January or February. The fluctuation in such wells ranges from 10 to 20 ft. in different portions of the basin. Irregularities superimposed on the broad periodic curve are the result of irregularity in the application of irrigation water.

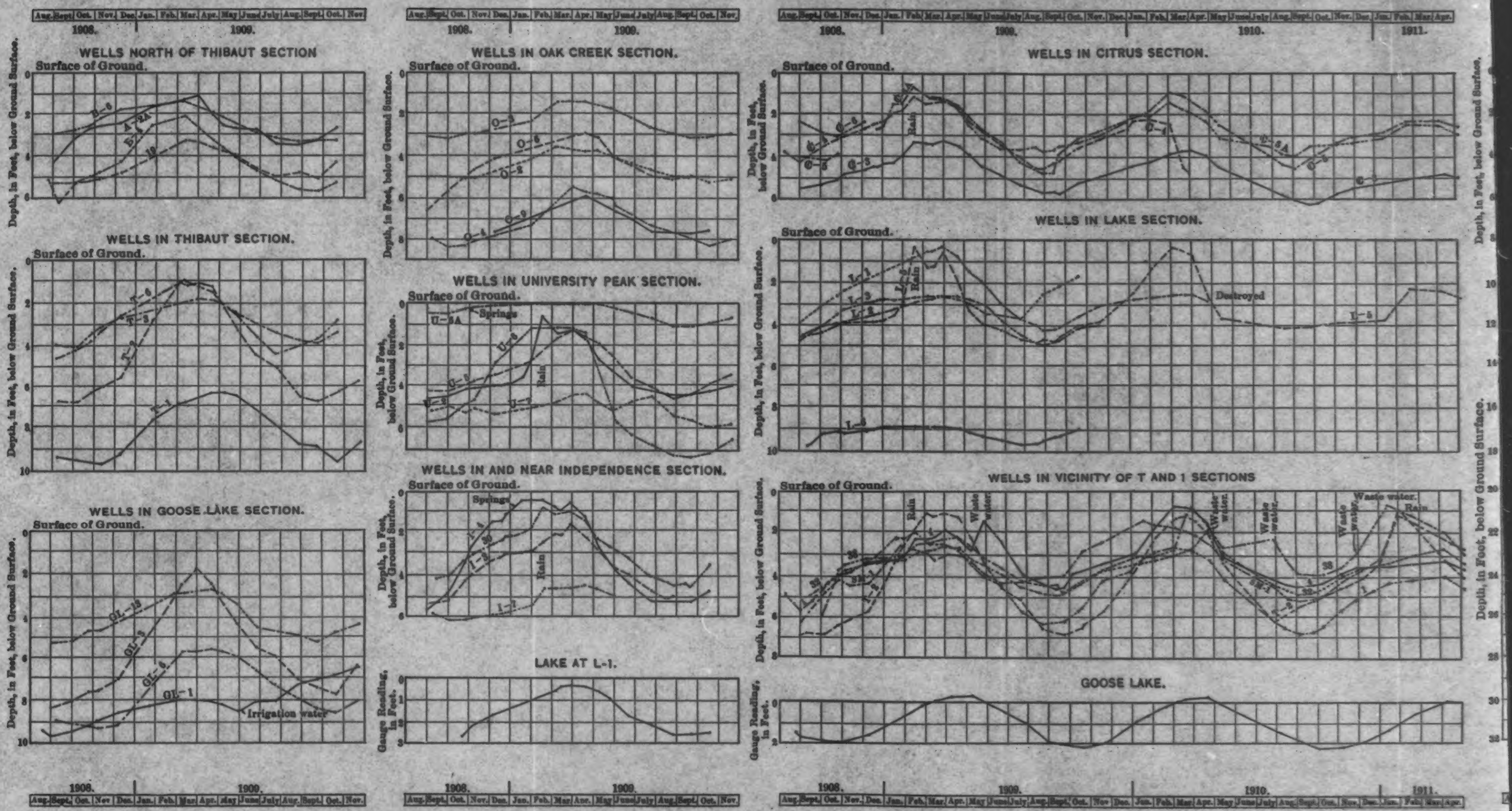
The fluctuation of the ground-water surface in various parts of the valley floor, other than at the eight wells already mentioned, is shown by 48 typical well records on Plate VI.

Permanent bench-marks were established at each well, and test holes from which measurements to the water surface could easily be made with a steel tape. Before observing, a weight is fastened at the end and the tape is chalked. The end of the tape is then submerged and the difference between the readings at the bench and at the water surface is the depth. Corrections are made in the office when the bench is not at the ground surface. Readings are to feet and hundredths.

Most of these wells are where the average depth to ground-water is less than 8 ft. The adjoining ground surface is more or less crusted with alkali, and, where the alkali is not too concentrated, several species of wild grass grow vigorously. The two lakes and Wells Nos. C-3 and 43 are in areas where the average depth to ground-water exceeds 8 ft., and desert conditions prevail.

The fluctuations observed in all the valley-floor wells are remarkably uniform. When platted, the observations give smooth and regular curves with an annual periodicity. The average time of occurrence of crests for wells in grass or alkali areas is March 28th; the troughs occur on September 20th, 6 months later. Heavy winter precipitation, or the proximity of springs, advances the crests into January or February, but, in the desert areas, the crest lags into April or May. The fluctuation between maximum and minimum levels in normally situated wells ranges from 1.5 to 4 ft. Wells which are near or below springs in the vicinity of intermittently occurring surface water have a greater range, which may reach 7 ft. The average fluctuation for 1908-09, as observed in 122 wells distributed generally over the valley floor, is 3.14 ft. This average represents normal conditions.

DIAGRAMS SHOWING FLUCTUATION OF GROUND-WATER SURFACE. TYPICAL OBSERVATION WELLS



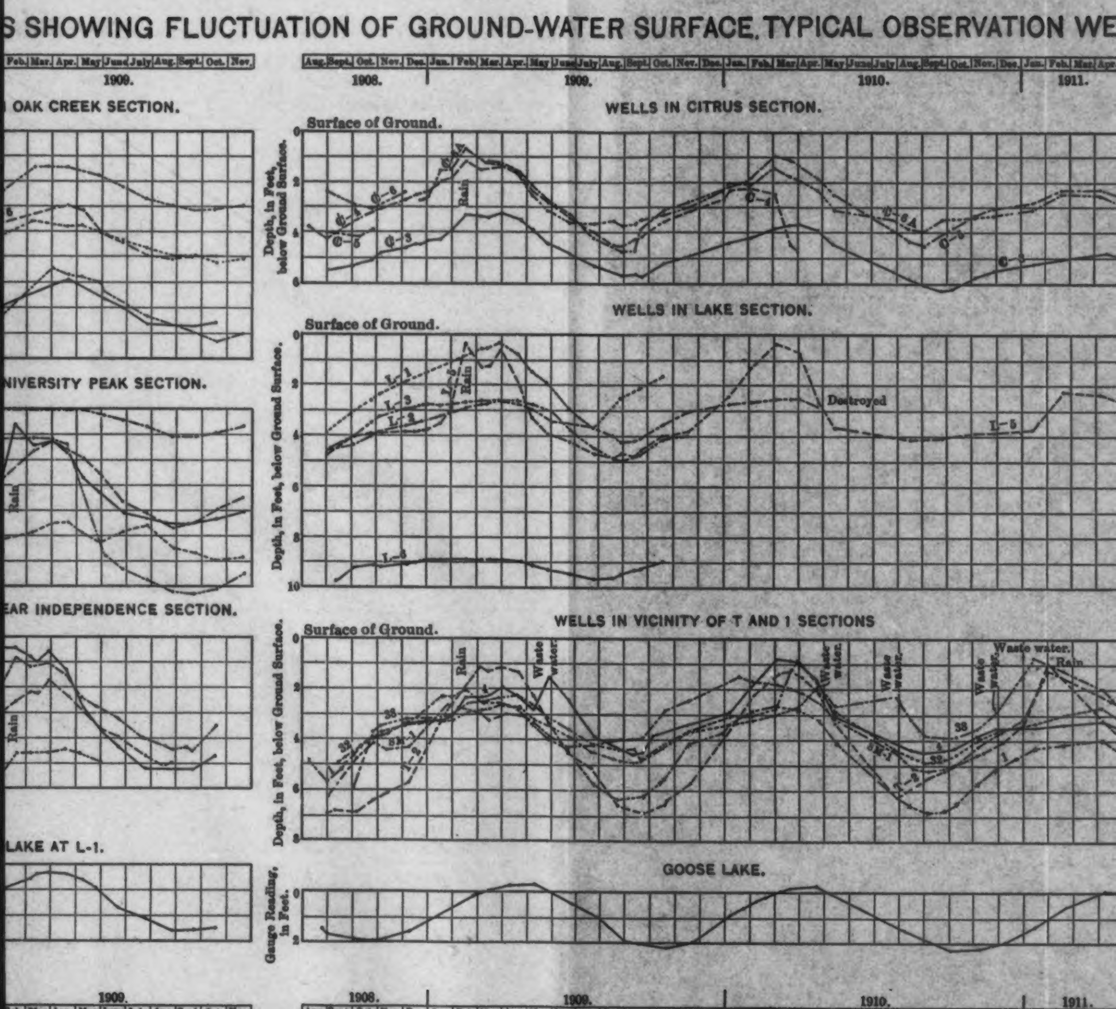
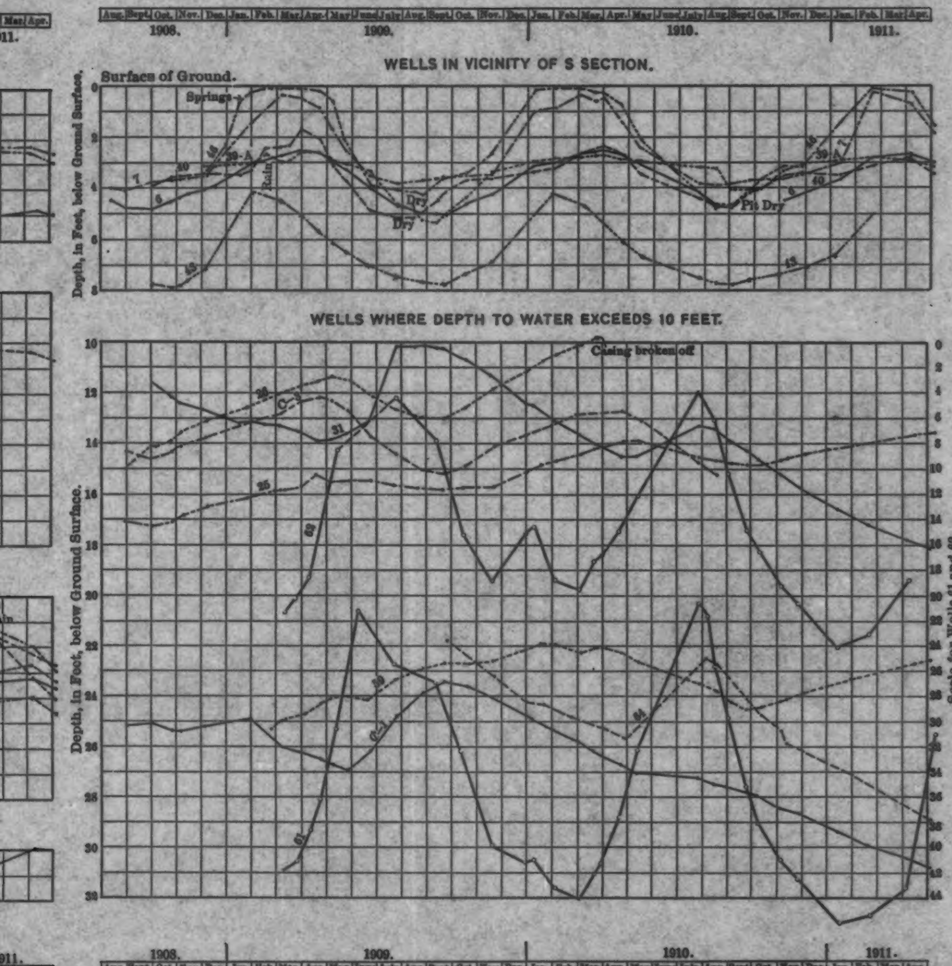

S SHOWING FLUCTUATION OF GROUND-WATER SURFACE. TYPICAL OBSERVATION WE
1909.
OAK CREEK SECTION.
NIVERSITY PEAK SECTION.
EAR INDEPENDENCE SECTION.
LAKE AT L-1.
1908.
1909.
1910.
1911.
WELLS IN CITRUS SECTION.
Surface of Ground.
Depth, in Feet, below Ground Surface.
WELLS IN LAKE SECTION.
Destroyed
WELLS IN VICINITY OF T AND 1 SECTIONS
Waste water.
Rain
GOOSE LAKE.
Gauge Reading, in Feet.

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1315.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.

WELLS IN INDEPENDENCE BASIN



Fluctuation of this type is due to evaporation from the soil and transpiration, processes which are active wherever there is capillary connection between the surface of saturation and the ground surface, or wherever gravity water or capillary water is within reach of plant roots. Two facts have led to this conclusion: (1) the area characterized by capillary connection between ground-water surface and ground surface and by accessibility of ground-water to plant roots is coincident with the area exhibiting this type of periodic fluctuation; and (2) the combined rates of evaporation from soil and transpiration, as observed experimentally, increase and decrease concurrently and in the same ratio with the fall and rise of the ground-water surface.

The first of these facts is indicated by the following observations: Surface incrustations of alkali are now known among investigators to be an indication of evaporation from the soil, and a growth of natural grasses certainly shows the presence of water within reach of plant roots. These manifestations are both strictly confined to valley-floor areas within which the periodic fluctuation is observed. There are valley-floor areas, however, within which the periodic fluctuation occurs, but which have a loose sandy surface devoid of alkali and vegetation. An examination of such areas shows that they are surrounded or bordered by meadow and alkali-crusted land, and further that maximum and minimum ground-water levels exhibit a lag in time of occurrence which varies with the distance from these adjoining lands. (See Plate VI, Well C-3 and Goose Lake.) The fluctuations in these desert areas do not originate within the areas themselves but in the near-by lands, from which they are propagated as annual waves. In general, average depths to ground-water exceed 8 ft. in desert areas but are less than 8 ft. in meadow or alkali lands.

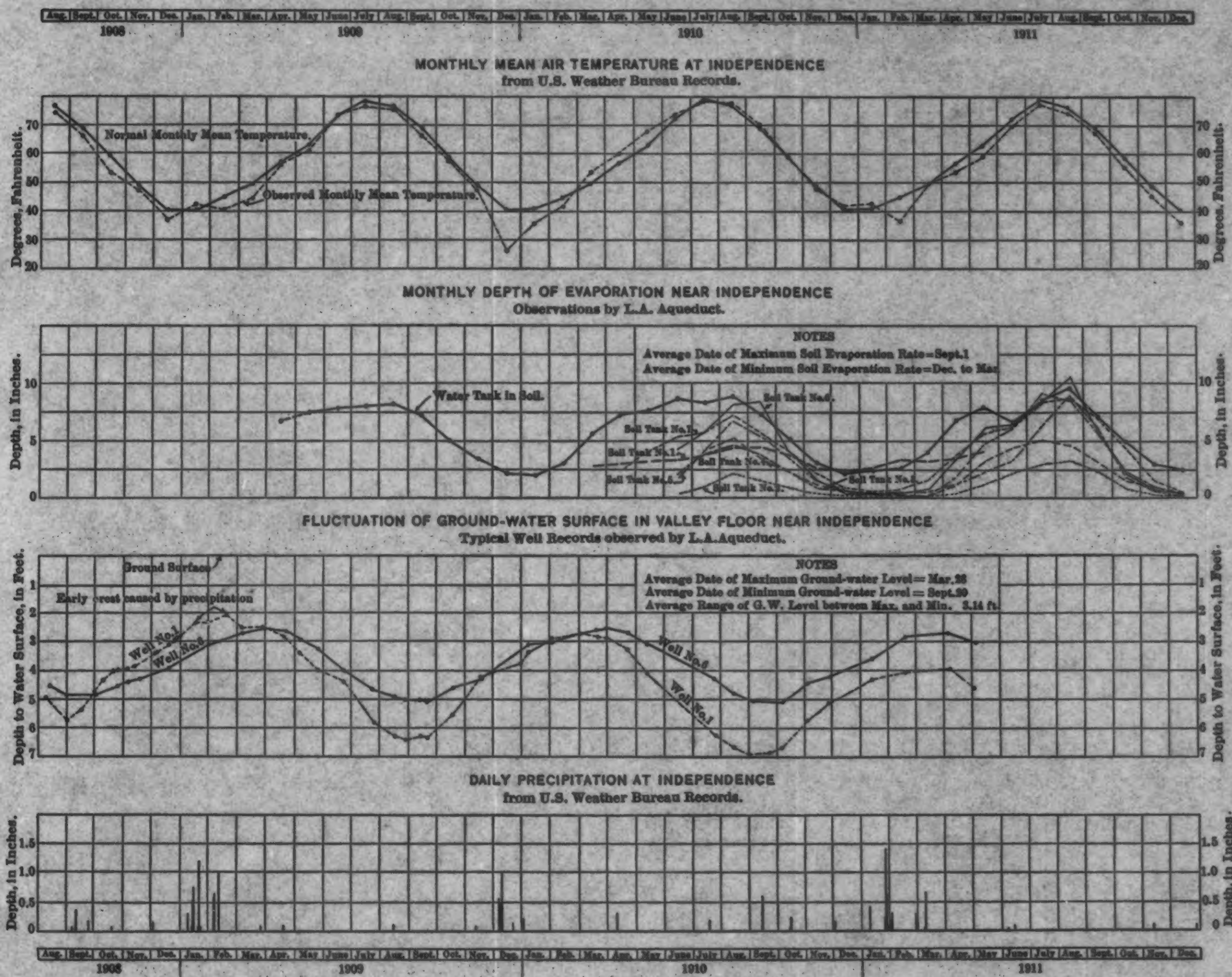
The second fact—that variations of soil evaporation and transpiration are similar to ground-water fluctuations—is indicated by the results of the experiments on evaporation from soil. The maximum rate of soil evaporation occurs about September 1st and the minimum from December 1st to April 1st. The lowest ground-water level occurs about September 20th and the highest level on March 28th (Plate VII). Thus the critical points in the curves of soil evaporation and ground-water fluctuation are practically coincident as regards time. Furthermore, although the curves are inversely related, their form is remarkably similar. The obvious conclusion is that ground-water fluctuations

in non-irrigated portions of the valley fill are the result of evaporation from the soil and transpiration.

Variations from the normal periodic curves occur for three causes: large precipitation, seepage from springs, and seepage from standing or flowing surface water. The infrequency of precipitation sufficient to raise the ground-water surface is shown on Plates VI and VII. It is practically a negligible factor in ground-water fluctuations. The springs at the upper edge of the outwash slope affect ground-water conditions in their vicinity by stimulating the annual rise and maintaining the ground-water level at a maximum during several months prior to March. (See Plate VI, Wells Nos. 46, U-6A, and I-4.) This results from the decrease in the rate of soil evaporation which allows the accumulation of their discharge in the surrounding soil at a greater rate than in adjoining areas where the rate of supply of underground water is less. Surface water has its source in large springs, waste from irrigation, and the flood waters of mountain streams. It occurs at various times and places, and cannot be considered as a permanent factor in ground-water fluctuations. (See Plate VI, Wells Nos. 4, 38, 39-A, 32, and GL-1.) The irregular fluctuations of ground-water on the outwash slope do not appear in the valley floor because of the relief afforded by the escape of water in springs at the upper edge of the grass land.

Ground-Water Losses.—Ground-water fluctuations within the valley floor consist primarily of the regular annual rise and fall produced by variation in the rate of evaporation. This is indicated by actual observations extending over 3 years, and confirmed by the persistency of various perennial plant species. Hence there must be overflow of ground-water from the valley fill of the region equal to the average inflow by percolation. The possible outlets would seem to be underflow southward through the valley fill, underflow by way of deep fissures, seepage and spring flow into the channel of Owens River, evaporation from spring waters, evaporation from damp soil, and transpiration from vegetation. The first two of these are eliminated by the geology and topography of the region. The slope of the ground-water surface in the valley fill opposite the Alabama Hills does not exceed 8 ft. to the mile, and the material is fine sand and clay, as indicated by the Southern Pacific Company's well at Lone Pine Station. Even if there is a movement of ground-water southward from the

TEMPERATURE, EVAPORATION FROM SOIL, PRECIPITATION AND FLUCTUATION OF GROUND-WATER SURFACE IN VALLEY FLOOR NEAR INDEPENDENCE.



region, it must be exceedingly slow, and it would be entirely intercepted by the alluvial fan of Lone Pine Creek, which has a groundwater surface higher than the valley fill to the north. The granitic formation of the Sierra Nevada and the granite core of the Inyo Mountains are complete barriers against the escape of underground waters through any formation but the valley fill. It has already been shown that there is no seepage flow into Owens River from the water supply of the region. Hence the outlets by evaporation, transpiration, and spring discharge into Owens River are all that remain to be considered.

Soil evaporation and transpiration will be considered, first, for irrigated lands, and second, for the general grass and alkali area of the valley floor. The quantity of water used in irrigating the 3 011 acres under cultivation in the region is about 72 sec-ft. of continuous flow for 6 months (Table 18), which is equivalent to a depth of 8.6 ft. over the whole area. The depth of transpiration from alfalfa during the irrigating season has already been computed as 3.43 ft., or 40% of the total volume used. There is also a small loss through evaporation from the soil during and immediately after irrigations, say 0.85 ft., or 10% of the total. The total loss by evaporation from the soil and by transpiration from irrigated areas, therefore, is 4.3 ft. in depth, or 18 sec-ft., of continuous flow.

The bases for computing the evaporation and transpiration loss from grass and alkali land are the soil-evaporation equations of Fig. 10 and the ground-water contours of Plate V. The equations were developed for a fixed ground-water surface, but they cover the periods from October 1st to March 31st and from April 1st to September 30th, which practically coincide with the observed periods of rising and falling ground-water. Hence, to cover the natural conditions of fluctuating ground-water surface, average annual depth to groundwater at a given point may be substituted in the equations instead of fixed ground-water depths. The average annual depth to groundwater, in feet, and the depth of evaporation, as determined from the equations for 1911, are given in Table 15 for the non-irrigated areas enclosed by the 3-ft. contour and between the 3- and 4-ft. and 4- and 8-ft. contours. The volume annually evaporating from the whole area enclosed by these contours is equivalent to a continuous flow for the year of 109 sec-ft.

TABLE 15.—TOTAL EVAPORATION FROM GRASS AND ALKALI LANDS IN THE VALLEY FLOOR.

(BASED ON 1911 DATA.)

Enclosing contours.	Area, in square miles.	Average depth to ground-water, in feet.	ANNUAL DEPTH OF EVAPORATION, IN INCHES.			Equivalent flow, in second-feet.
			Summer.	Winter.	Total.	
3 Ft.	11.89	2.5	36.5	5.2	41.7	36.6
3-4 Ft.	17.66	3.5	29.6	4.0	33.6	43.7
4-8 Ft.	25.04	5.5	15.6	0.2	15.8	29.1
Totals.	54.59					109.4

The water of Blackrock and Hines Springs, and of the small springs along the upper edge of the valley floor, spreads out in many shallow lake basins before reaching Owens River. The loss by evaporation from the surface of these lakes is large. Estimates based on the area of water surface exposed and the evaporation from water in the shallow pan in soil indicate that about 50% of the flow of these springs thus escapes into the atmosphere. As the combined flow is 31 sec-ft., the loss by evaporation from the free water surface is 15 sec-ft. The portion of the remainder which does not flow into Owens River percolates into the soil and escapes by evaporation from the soil and by transpiration.

Two springs derive their waters from percolation and discharge directly into Owens River; these are Upper and Lower Seeley Springs. Their combined average flow is 11 sec-ft. In addition, the Blackrock Springs discharge an average of 7 sec-ft. into the river during November to March, inclusive, which is equivalent to a continuous flow of 3 sec-ft. The total discharge into the river from springs, therefore, is 14 sec-ft.

The grand total ground-water losses, therefore, are 156 annual sec-ft., of which 127 sec-ft., or 81%, is by soil evaporation and transpiration.

RATE OF RECHARGE BY PERCOLATION.

From Precipitation.—All portions of the region receive precipitation, but there is wide local variation in the quantities which enter the ground and percolate downward to the surface of saturation. The impervious rock surfaces of the high mountain drainage areas shed

all precipitation which they receive except that lost by evaporation, but accretions to the ground-water from precipitation on the remaining areas of the region are of considerable importance.

Conditions are exceptionally favorable for percolation on the intermediate mountain slopes. As has been stated, the formation is very porous, and practically none of the run-off reaches living streams. All precipitation but that lost by evaporation, therefore, can be considered as percolating downward to the surface of saturation and becoming a permanent addition to the ground-water supply. Snow is practically all melted before May 15th, so that the period of direct exposure to evaporation is not as long as at higher levels, although the rate is greater. Evaporation losses from the moist soil are very small. Before it is melted, the snow blanket protects the soil surface, and by its gradual and uninterrupted melting it fills the capillary spaces to considerable depth, so that gravity water passes downward rapidly. When the snow disappears the rapid drying of the soil surface soon interrupts upward capillary movement, thus preventing further evaporation loss and allowing the percolating water to reach the surface of saturation. In view of these facts, the percolation factor is regarded as being about 0.75 for the more elevated areas receiving approximately 20 in. of precipitation. Less favored areas were assigned smaller factors after a study of their individual characteristics.

The results of computations of the total quantity of percolating water yielded by the intermediate mountain slopes are shown in Table 16. The method was to determine the mean seasonal precipitation at the center of area of each triangular subdivision, multiply this by the area in square miles, and apply a percolation factor. The area of each subdivision and the horizontal and vertical position of its center were obtained from Table 3. Diagrams 7, 8, 10, 11, 13, and 14 on Plate II were used in determining the depth of precipitation. The values differed slightly, as read from the altitude and distance diagrams, and the average was adopted as the most reliable. The total volume of precipitation on the 29.4 sq. miles of intermediate mountain slope is 27 580 acre-ft., of which 19 700 acre-ft. is a permanent addition to the underground water supply of the region. Expressed as a continuous flow, the total percolation from this area is 27 sec-ft.

The outwash slopes yield to the underground supply a much smaller volume of water, which is derived principally from slopes above

TABLE 16.—PERCOLATION FROM PRECIPITATION UPON INTERMEDIATE MOUNTAIN SLOPES OF INDEPENDENCE REGION.

(Mean seasonal values.)

Adjoining high mountain drainage area.	Depth of Precipitation on Center of Area, in Inches.*			Volume of precipitation on area in acre-feet.	Percolation factor.	Quantity of Percolation.	
	A.	B.	Average.			Volume, in acre-feet.	Discharge, in second-feet.
Taboose Group of Precipitation Gauges.							
Tinemaha	22.2	17.8	20.0	2 320	0.75	1 740	2.4
Red Mountain	23.6	17.8	20.7	2 620	0.75	1 970	2.7
Taboose	22.2	16.5	19.4	4 080	0.75	3 060	4.2
Goodale	18.5	20.0	19.2	2 350	0.75	1 760	2.4
Division	19.9	15.5	17.7	900	0.70	630	0.9
	....	....	....	12 270	....	9 160	12.6
Oak Group of Precipitation Gauges.							
Sawmill	18.2	18.4	18.3	1 290	0.70	900	1.2
Thibaut, North Fork	18.2	18.9	18.6	580	0.70	370	0.5
Thibaut, South Fork	14.5	15.0	14.8	60	0.60	40	0.1
Oak, North Fork	16.4	14.2	15.3	2 950	0.60	1 770	2.4
Oak, South Fork	16.4	15.8	16.1	880	0.70	620	0.9
Little Pine	17.7	15.8	16.8	1 810	0.70	1 270	1.8
Pinyon	18.7	20.8	19.8	3 050	0.75	2 290	3.2
	....	....	....	10 570	....	7 260	10.1
Bairs Group of Precipitation Gauges.							
Symmes	13.4	17.0	15.2	340	0.70	240	0.3
Shepard	12.7	12.4	12.6	650	0.65	420	0.6
Bairs, North Fork	12.4	10.8	11.6	300	0.65	200	0.3
Bairs, South Fork	14.8	12.0	13.4	860	0.70	600	0.8
George	15.8	15.8	15.8	1 760	0.70	1 230	1.7
Hogback	15.2	13.6	14.4	830	0.70	580	0.8
	....	....	....	4 740	....	3 270	4.5
Grand total	....	....	....	27 580	....	19 690	27.2

* Depth of precipitation as obtained by the precipitation-altitude diagram is given under A; as obtained by the precipitation-distance diagram under B. The average is taken for use in computations.

the 5 500-ft. contour. Precipitation occurs as snow less often here than on the higher slopes, and usually melts within a few days after falling. The capillary water in the upper layers of the soil thus has opportunity to evaporate after each storm, and it is only when several storms occur in succession that there is enough percolating water to

penetrate the ground beyond possibility of return. The long dry summer and the desert conditions draw all moisture from the ground to considerable depths, and the progress of percolating waters is slow because the capillary spaces must be refilled. Test pits dug in the region of the 4 500-ft. contour, 10 days after a series of storms, showed a penetration of capillary water to a depth of 4 ft. and the entire absence of gravity water. The total precipitation from these storms at this point was about 3.5 in., which, with a 28% available pore space, would represent 1 ft. of completely saturated soil. Considering the evaporation losses, it is not surprising that there was no gravity water within the depth of penetration observed. Observations made at higher elevations after this storm showed gravity water in considerable quantity at a depth of 12 ft. Percolation factors varying from zero to 0.60 were assigned to the several zones of the outwash slope as a result of these field observations.

The results of computations for the total quantity of percolating water yielded by the outwash slopes are shown in Table 17. The whole area of 165 sq. miles was divided into zones lying between contours at 500-ft. intervals from about 4 000 to 6 500 ft., and the zones in turn were divided into groups corresponding with the precipitation gauges. The method of computation was to average the precipitations for adjacent contours obtained from Diagrams 7, 8, 10, 11, 13, and 14 of Plate II. These averages represented the average precipitation for each zone in each group, and, when multiplied by the area and the percolation factor, gave the quantity of percolating water which reached the permanent ground-water level. The total annual precipitation on the outwash slopes is 62 000 acre-ft., of which 16%, or 9 800 acre-ft., is effective percolating water. Expressed as a continuous flow, the volume of percolating water amounts to 13.4 sec-ft.

Throughout the valley floor the surface of saturation is so close to the ground surface that capillary connection is maintained during most of the year, and percolation from precipitation is rapid. The depth of penetration is usually slight, however, because precipitation in single storms is small. Several storms in succession or a warm rain on snow will result in a rise of ground-water, but the total average ground-water supply from this source does not exceed 4 sec-ft.

Direct percolation from precipitation, therefore, furnishes a grand total of 44 annual sec-ft. to the underground supply of the basin.

TABLE 17.—PERCOLATION FROM PRECIPITATION UPON OUTWASH SLOPES OF THE INDEPENDENCE REGION.

(Mean seasonal values.)

Contours bounding precipitation zones.	Area of zones, in square miles.	Depth of precipitation, in inches. On contours.	Depth of precipitation, in inches. On zone.	Volume of precipitation on zone, in acre-feet.	Percolation factor.	Quantity of percolation. Volume, in acre-feet.	Quantity of percolation. Discharge, in second-feet.
Taboose Group of Precipitation Gauges.							
Grass-4 500.........	28.07	6.0, 7.7	6.8	10 180	0.00	0	0
4 500-5 000.........	8.54	7.7, 9.0	8.4	3 830	0.10	380	0.5
5 000-5 500.........	8.14	9.0, 10.8	9.9	4 300	0.20	860	1.2
5 500-6 000.........	5.56	10.8, 12.9	11.8	3 500	0.35	1 220	1.7
6 000-6 500.........	3.42	12.9, 15.2	14.0	2 550	0.60	1 530	2.1
	53.73	....	....	24 360	....	3 990	5.5
Oak Group of Precipitation Gauges.							
Grass-4 500.........	24.57	4.8, 5.9	5.4	7 080	0	0	0
4 500-5 000.........	11.78	5.9, 7.3	6.6	4 150	0.05	210	0.3
5 000-5 500.........	9.23	7.3, 9.1	8.2	4 040	0.15	610	0.8
5 500-6 000.........	5.82	9.1, 11.3	10.2	3 170	0.30	950	1.3
6 000-6 500.........	4.82	11.3, 13.6	12.4	3 190	0.50	1 600	2.2
	56.22	....	....	21 630	....	3 370	4.6
Bairs Group of Precipitation Gauges.							
Grass-4 500.........	19.56	3.3, 4.0	3.6	3 760	0	0	0
4 500-5 000.........	11.68	4.0, 5.2	4.6	2 860	0	0	0
5 000-5 500.........	9.76	5.2, 6.9	6.0	3 120	0.10	310	0.4
5 500-6 000.........	9.23	6.9, 8.6	7.8	3 840	0.25	960	1.3
6 000-6 500.........	5.11	8.6, 10.3	9.4	2 560	0.45	1 150	1.6
	55.34	....	....	16 140	....	2 420	3.3
Grand total....	165.29	....	....	62 130	....	9 780	13.4

From Stream Channels.—The most important source of underground water in a desert region is percolation from stream channels. This process is continuous from perennial streams, although it varies with the discharge of the streams and the temperature, as previously indicated. Beneath each stream channel as it crosses the outwash slope is a "ridge" of ground-water rising from the general plane of satura-

tion. The inclination of the slopes of this ridge and the breadth of its base vary periodically with the stage of the creek and the time of year. There is complete saturation within its slopes and a movement of gravity water toward the general ground-water surface. A considerable quantity of water also percolates from intermittent streams.

Percolation from stream channels in the Independence Basin is confined to the creeks draining mountain canyons. There are 17 of of these streams, 11 of which are perennial throughout their channels, 5 are perennial over the upper portion of their channels only, and 1 (Dry Canyon) is entirely an underground stream. The surface flow of the two most northerly of these streams, Tinemaha and Red Mountain Creeks, discharges northward across the Poverty Hills into the Bishop-Big Pine region, but the percolation from their channels is tributary to the Independence Basin. The channels of streams entirely within the basin are continuous from their canyons to the U. S. Geological Survey gauging stations, below which they divide, irrigation ditches carrying all the flow except during the high-water period of wet years, when the excess passes down the natural channels. The problem is thus divided into the determination of percolation above and below the Government gauging stations. The first subject has already been discussed at length and need not be considered here in detail. An inspection of Tables 4 and 5 shows that for the creeks from Taboose to Hogback, inclusive, the total 21-year average discharge at the mouths of the canyons is 130 sec-ft. and at the Government gauging stations 84 sec-ft. If the flow of 2 sec-ft. from Spring No. 2 on Division Creek is included with the canyon discharge, the percolation loss above the Government gauging stations is 48 annual sec-ft. To this should be added 6 sec-ft., as indicated by the diagrams, for Tinemaha and Red Mountain Creeks, making a total of 54 sec-ft.

The quantity lost below these stations is not so easily determined, on account of the numerous channels and irregular flow. Estimates were made on each creek, based on the length of main channel and distributing ditches outside of irrigated areas. The loss per mile was assumed to be the average annual loss per mile for the upper channel of the creek, and the total percolation loss from stream channels below the Government stations was estimated at 25 sec-ft. of continuous flow. This estimate does not include percolation from waste irrigation water or surplus creek water which has passed east of the ranches.

The grand total addition to the underground supply derived by percolation from stream channels, therefore, is 79 annual sec-ft.

From Irrigation.—Irrigation has been practised throughout this region, in connection with farming, for at least 30 years, and is a permanent factor in the underground water problem. The total area under systematic irrigation is approximately 3 000 acres, divided into a number of isolated ranch groups which depend on the mountain creeks for their supply. Oak Creek, the largest of these streams, supplies about 45% of the whole area. The remaining area is divided among eight creeks and the Stevens ditch, which during the period of observation has been largely supplied by the surplus flow of the creeks. The acreage irrigated from each source is given in Table 18. About 50% of this land was originally desert, lying along the lower margin of the outwash slope, and is very porous. The remainder lies in the valley floor, where permanent ground-water is within reach of plant roots and where clay soils predominate. The location of the several areas is shown on Plate V. Alfalfa and grain are irrigated by

TABLE 18.—ESTIMATED NET VOLUME OF WATER USED FOR IRRIGATION IN THE INDEPENDENCE REGION DURING 1909.

Source of supply.	Area irrigated, in acres. *	Duty of water per acre for season, acre-feet. †	Total Volume of Water Used. Acre-feet.	Total Volume of Water Used. Second-feet for 6 months.
Taboose Creek	(170)	(12)	2 040	5.6
Goodale Creek	(110)	(16)	1 760	4.9
Division Creek	(80)	(16)	1 280	3.5
Sawmill Creek	(90)	(16)	1 440	4.0
Oak Creek, ranch No. 1	109	7.22	790	2.2
Oak Creek, ranch No. 2	49	15.40	758	2.1
Oak Creek, ranch No. 3	155	2.80	435	1.2
Oak Creek, ranch No. 4	260	2.34	609	1.7
Oak Creek, ranch No. 5	38	16.40	623	1.7
Oak Creek	(80)	(16)	1 280	3.5
Oak Creek	(100)	(5)	500	1.4
Oak Creek	(560)	(3)	1 680	4.6
Little Pine Creek	(300)	(14)	4 200	11.6
Symmes Creek	(160)	(5)	800	2.2
Shepard Creek	(280)	(12)	3 360	9.3
George Creek	(160)	(12)	1 920	5.3
Stevens ditch	(310)	(8)	2 480	6.9
	3 011	....	25 955	71.7

* Areas in parentheses obtained from approximate field observations; other areas obtained by careful field measurement.

† Figures in parentheses assumed from observations on Oak Creek ranches.

flooding, and corn by the furrow method. Three crops of alfalfa are raised each year, and the irrigating season extends from about April 15th to October 15th, although some farmers irrigate 9 months in the year. Grain is irrigated early in the season, and corn late, so that the water is continually used. In most places the use of water is lavish, and no attempt is made to economize it or even to apply the quantity best suited to the crop and soil conditions.

A basis for determining the percolation from irrigation is a knowledge of the duty of water, or the quantity of water used in maturing a given area of crop. This was obtained in 1909 by carefully measuring the quantity of water used daily during the irrigating season on five typical ranches which derived their supply from Oak Creek. On ranches where there was a continual waste from irrigation the surplus water was also measured. Areas in crop were obtained from a careful stadia survey of each ranch.

With conditions on these typical ranches in mind, an examination was made of all other ranches in the region, and values were estimated for the duty of water on each. The number of acres irrigated and in crop was also determined approximately by reference to subdivisions of the public survey. From these data the volume of water used for irrigation was determined, as shown in Table 18. The total volume used during the 6 months, April 15th to October 15th, is about 26 000 acre-ft., equivalent to a continuous flow of 72 sec-ft. during the period. When spread out over 3 010 acres, this represents an average depth of 8.6 ft. This result probably represents an average practice throughout the Owens Valley, for the duty of water measured by the Reclamation Service during the season of 1904 on two typical ranches near Bishop was 7.11 and 9.17 acre-ft. per acre.

The distribution of this water, as regards evaporation, transpiration, and percolation beyond the reach of plant roots, is the next step in computing the ground-water supply from this source. Direct evaporation is relatively small, for the water when spread out over the fields is shaded by the crop and sinks rapidly into the ground. Probably 10% would cover this loss. The transpiration loss from alfalfa during the irrigation season has already been computed as 3.43 ft. depth of equivalent water, or 40% of the average volume of water applied to crops. The transpiration loss from corn and small grains is probably less in this locality, but the direct evaporation loss is greater. There-

fore 50%, or 4.3 ft. depth, represents the quantity of water applied in irrigation in this region which is absorbed by the atmosphere. The other 50% is a permanent addition to the ground-water supply, and is equivalent to a continuous flow of 18 sec-ft. throughout the year.

From Flood Water.—The quantity of percolation from surplus creek water, which spreads out over the valley floor to a greater or less extent, is difficult to determine. Of the 84 sec-ft. average flow at Government gauging stations, 61 sec-ft. are disposed of in channel percolation and irrigation. Possibly 5 sec-ft. of the remainder reach Owens River. This leaves 18 sec-ft. to be divided between evaporation and percolation in the flats between the ranches and the river. The area flooded averages about 5 sq. miles during June and July. The loss by evaporation during this period from a shallow pan in soil was about 24.5 in., and, as the conditions are similar, this represents approximately the loss from shallow flood water. The volume expressed as a continuous flow for two months is 55 sec-ft., or, for a year, 9 sec-ft. The other 9 sec-ft. can be assumed to represent the percolation from this flood water. It is not a permanent addition to the ground-water supply, however, for the surface of saturation is only a few feet below the ground surface in this area, and evaporation from damp soil and transpiration from natural vegetation soon reduce the ground-water surface to its normal position.

Summary of Percolation.—The four sources of ground-water are percolation from direct precipitation, from stream flow, from irrigation, and from flood water in the valley floor.

The first of these yields about 44 annual sec-ft., of which 61% is from the intermediate mountain slopes, 30% from the outwash slopes, and 9% from the valley floor. Percolation from streams yields about 79 annual sec-ft., of which 68% is above Government gauging stations and 32% below. Irrigation yields 18 annual sec-ft. and flood waters in the valley floor 9 annual sec-ft.

The grand total ground-water, therefore, is 150 annual sec-ft., of which probably 75% reaches the deeper strata of the valley fill. The rate of recharge of the basin, as thus determined, differs by less than 4% from the ground-water loss previously computed. The reliability of the data is thus confirmed as well as the correctness of the assumptions.

Safe Yield.

Thus far, this paper has presented conditions as they are found to exist in a natural state. The problems which the engineer has to solve are those connected with the artificial extraction of water from underground reservoirs. First among these is the determination of the safe annual yield or the limit to the quantity of water which can be withdrawn regularly and permanently without dangerous depletion of the storage reserve. A second problem which naturally accompanies the first is the devising of methods for increasing artificially the safe annual yield of reservoirs which are apparently already developed to the limit. The writer will outline his ideas, in the hope that they will suggest a constructive line of discussion which will lead to a better understanding of these subjects.

It is obvious that water permanently extracted from an underground reservoir, by wells or other means, reduces by an equal quantity the volume of water passing from the basin by way of natural channels. This is illustrated by the commonly recognized fact of the drying up of springs and cienagas as the result of heavy pumping. The theoretical limit for safe draft, exclusive of return water, therefore, is the average rate of recharge for a basin. The practical limit, however, depends on the relation of draft to storage capacity, within economic pumping limits. Where the storage capacity is very large as compared with annual draft, the theoretical and practical limits should nearly agree, as the storage reserve can be drawn on in periods of protracted drought. For basins with comparatively small storage capacity, the practical limit will be less than the theoretical. Draft computations may be made with the mass-diagram as ordinarily used in surface storage problems. Storage capacity is determined from the area of water-bearing material, limiting depth for economic pumping, and percentage of voids capable of depletion. The supply is the quantity of water annually absorbed by the porous material of the basin. This may be determined each year by methods similar to those used in the Owens Valley studies.

The draft thus obtained, however, is not the safe yield of the basin, for there are always certain residual losses which cannot be entirely prevented, such for instance as soil evaporation from cienaga lands. These residual losses must be ascertained and deducted from the gross draft. The quantity thus obtained may be persistently with-

drawn from the basin without causing general depression of the water plane to the point where pumping operations must cease for economic reasons.

The determination of residual losses presents difficult problems. Some of the conditions which are responsible for these losses are the following:

1.—The elevation of the impervious rim at the outlet being less than the elevation of the water plane in the lowest depression of the basin, thus allowing ground-water to escape as underflow. The quantity of water thus dissipated depends on the transmission capacity and area of the porous material overlying the rim and the available head. In most cases the volume of water thus lost is relatively very small.

2.—The outlet of springs being at a considerably lower elevation than the general water plane of the basin in the outlet region. Such a condition may exist where an arroya has cut a channel through the impervious rim at a point where the surface falls away rapidly down stream. Such losses are also relatively small.

3.—The occurence of water under Artesian pressure in underlying strata of porous material confined between more impervious layers of fine sand or clay. This is the least recognized, but yet the most important, cause of residual ground-water loss. The effect of Artesian pressure is to force moisture through pores or fractures in the impervious capping and thus maintain a permanent ground-water plane near the surface. The water continually supplied from below is disposed of either by evaporation from the soil surface and vegetation or by escaping at the surface in springs or seepages. These losses persist as long as the Artesian pressure is sufficient to force the water through the overlying strata. The volume of these losses during any period of one or more years bears a functional relation to the average Artesian pressure during the same period. The writer states these conclusions as the result of a careful study of records and conditions in a number of differently situated Artesian basins.

These conclusions are well illustrated by the following facts of common knowledge. First, consider the result of abnormally large precipitation for a period of one or more seasons. The ground-water accretions exceed the losses from the basin, the excess water accumulates in the voids of the porous gravels surrounding the confined strata,

and the free ground-water surface rises throughout the basin. Within the area of confined gravels hydrostatic or Artesian pressure increases. A greater quantity of water is forced through the overlying strata, not only over the area already moist, but from a circumscribing zone within which the pressure was previously insufficient to maintain a shallow ground-water surface. The observed result, therefore, is increased spring and seepage flow and an enlarged area of moist cienaga land from which evaporation occurs. Second, assume a series of dry years. Ground-water storage is depleted, the free ground-water plane falls, Artesian pressure decreases, and less water is forced through the overlying strata. The observed result is decreased spring and seepage flow and shrunken evaporating area. The latter occurs, because, for the new conditions, the rate of evaporation from the outer zone of moist soil exceeds the rate of supply from below. The accumulation of water in the soil is drawn on, lowering the water level to the limit of capillary action. A similar result occurs where relief is afforded to Artesian pressure by the drilling of many deep wells drawing from Artesian strata. In some of the Southern California Artesian basins, which formerly possessed cienaga lands, relief of Artesian pressure by wells and heavy pumping has dried up such lands.

The importance of residual losses, due to Artesian pressure, is forcibly shown by the Owens Valley studies, where it was found that in a natural state 81% of the total yield of the Independence Basin was lost by evaporation from soil and vegetation. Similar conditions, in a slightly less degree, existed in many of the Southern California basins before ground-water development was undertaken. The increased pumping of the last 10 or 15 years has eliminated evaporation losses almost entirely in some of the smaller basins. In the larger basins, such as the San Bernardino Valley and Coastal Plain, the reduction has not been as great, having ranged from 30 to 50% of original evaporation losses in the various basins. In these basins evaporation losses formerly represented from 50 to 75% of the total ground-water supply. Hence, the residual evaporation losses to-day represent from 15 to 35% of the total ground-water supply for the large Southern California basins, and can be said to average 25 per cent.

The quantitative determination of the residual losses from an underground reservoir can only be made after a detailed study of the local

conditions. The factors to be considered are the topography and geology of the basin and its porous filling, the distribution and type of sources of percolating water, the rate of evaporation and transpiration, the depth of capillary action and the character of the soil within the evaporating area, the necessity for irrigation and the value of overlying lands for agricultural crops, the present or probable ultimate method of development of water, and the present or probable future use. The general condition favorable for small residual losses is the possibility of eliminating the evaporating area by lowering the water plane below the reach of capillary action. This may be done by the relief of Artesian pressure, by shallow pumping within the evaporating area, or by drainage. The first of these methods would result in reduced pressures in existing wells in the Artesian area and possibly lowered water levels in the back-water zone, especially if the ground surface has a steep slope. Shallow pumping and drainage, on the other hand, may be physically impractical or prohibitive in cost. Their success depends largely on the existence of shallow water-bearing strata from which water can be readily drawn.

There are three cases which arise in the determination of residual losses: first, a basin already fully developed or suffering from overdraft; second, a basin where the supply is partly developed; third, a basin entirely undeveloped. The first case can be recognized by inspection of the present or former evaporating area in connection with local confirmatory evidence and past records of water levels, yield of wells, etc. The residual losses may be ascertained from observations and measurements of existing conditions. The second and third cases require assumptions as to the method or combination of methods by which residual losses will be reduced to a minimum. These assumptions should be made after a careful study of local physical conditions and the probable future use of the water. The next step is the determination of existing evaporation losses, by contouring the ground surface and water plane, and ascertaining the soil evaporation losses for various depths to ground-water. The final step, namely, the determination of the percentage by which existing losses will be reduced by future development, is as yet largely a matter of judgment. Having arrived at some definite value, however, the residual losses due to evaporation can be computed, and, when combined with the losses from underflow or deep springs, the total quantitative result is obtained.

The thorough investigation of residual losses is essential in any determination of safe yield, and for this reason the writer has discussed the subject in detail.

Passing now from the determination of safe yield as limited by existing conditions to a discussion of methods for increasing safe yield artificially: Obviously, to accomplish the latter, either the rate of recharge of the basin must be increased or the percentage of unused water escaping from the basin must be decreased. Practical methods which suggest themselves are:

(1) Reduction of residual losses to the lowest possible quantity;
(2) Elimination of needless waste of underground water; and
(3) Increased absorption of surface flood waters.

The subject of residual losses has already been discussed at length. The writer wishes to emphasize the fact, however, that a basin has not been fully developed as long as the evaporating area persists in years of drought. The evaporating area is fully as important a criterion of the relation of withdrawals to supply as the water plane. A falling water plane does not of itself indicate overdraft. It is only when a rapid shrinkage and disappearance of the evaporating area accompanies a falling water plane that dangerous overdraft is indicated. Hence, in a closed Artesian basin from which the evaporation area has not disappeared, a greater yield may be obtained, provided an intelligent plan of development is followed.

A very common source of needless waste is from Artesian wells which are allowed to flow when the water is not in use. The wastefulness of this practice is so evident that it need not be discussed in this paper. Its continuance is made possible by lack of recognition of the ultimate effect of the practice among the owners of such wells. The writer feels that it is every engineer's duty and privilege to assist in guiding aright public opinion in matters of common interest, and suggests the subject of conservation of Artesian water as being pertinent in many communities.

The practicability of increasing the ground-water supply by bringing about greater absorption of flood water has been demonstrated in a number of California basins. The most extensive work of this kind is probably that done on the alluvial cone of Santa Ana River in the San Bernardino Valley. The method there used is to divert the flood

water of Santa Ana River in contour ditches from which it is distributed into smaller ditches which in turn subdivide, until finally the water spreads out over the porous alluvial gravels in a thin sheet and is absorbed. During the past few years the volume of water thus stored has averaged 12 000 acre-ft. annually, costing about 15 cents per acre-ft., including interest on the cost of permanent works. The work is capable of further expansion. The ultimate limit will be the ability to handle the violent floods, which are of frequent occurrence. The problem is not that alone of controlling the water, but of disposing of silt. The water is normally clear and is free from silt soon after the flood crest passes. Flood water, however, carries great quantities of silt, which deposits as soon as the velocity is checked. This forms an impervious layer of slime which seals the gravels and must be broken up and eventually removed in order to use the same spreading ground continuously.

The conditions on most California streams tributary to closed basins correspond to the Santa Ana. The writer is of the opinion that complete absorption of even ordinary floods on these streams cannot be brought about without temporary surface storage. This must be accomplished either by utilizing storage sites in the stream channel or by construction of contour levees on the alluvial cone. The purpose of such reservoirs would be to act both as settling basins and as temporary storage sites. From these reservoirs the clear water would be released and brought into contact with the absorbent gravels by any method which proved most efficient under the local conditions.

In conclusion it may be said, first, that the rate of recharge of underground reservoirs of the closed-basin type is a definite quantity capable of measurement with a fair degree of accuracy; second, that safe yield is a quantity less than the rate of recharge, its quantity depending on the available storage reserve of the basin and residual ground-water losses; third, that under certain circumstances it is possible to increase the safe yield of a fully developed basin.

DISCUSSION

JAMES OWEN,* M. AM. SOC. C. E.—Although Mr. Lee's paper is confined somewhat to underground water supplies in the western part of the United States, a great deal of investigation has been done and a great deal of money has been spent and wasted on such supplies in the eastern sections and in localities near New York City, and it seems that, if it were possible in the future to define certain rules and principles governing the question of getting water from the ground, it would be better for everybody. Mr. Owen.

The section around New York has a variety of topographical and geological conditions. According to the speaker's idea, the underground supplies can be defined under about four heads, that is, the original sandstone deposition, prior to the volcanic upheaval; the volcanic upheaval of gneiss in Westchester County and New York, and the trap in New Jersey; the glacial and post-glacial depositions, and, incidental to these, the terminal moraines; and finally, in lower New Jersey, what might be called the post-tertiary deposition. The first and the last, the original sedimentary deposition of the sandstone and the post-tertiary deposition, have fairly well-defined radii of supply.

It is well known that water can usually be obtained by driving through sandstone. In New Jersey, if one drives through certain strata, water can be obtained, and an interesting incident was shown when Asbury Park, N. J., desired a water supply and was advised by Professor Gay, State Geologist of New Jersey 30 years ago, to go down 800 ft., at which depth all the necessary water could be obtained. On trial, water was found within 40 ft. The final result has been that all through that shore territory the supply of water has been ample, good, and economical. In this section, however, the main consumption has only lasted about 4 months in the year, and during the other 8 months, the disposal of the underground supply would have failed to be profitable for steady communities.

In the sandstone formation, a good and reliable supply can always be obtained, subject to the chemical and natural properties of the water. In New Jersey, wells have been sunk 400 or 500 ft. below tidewater and ample water has been found, but its chemical constituents have made it rather detrimental for public use. In one case, where the well was put down 400 ft., the deposition of lime was about 8 in. in 4 months. That, of course, debarred its use for public purposes. The water, however, was storage water, and the slow percolation through the sandstone had not allowed for a free flow; consequently, the chemical deposition ensued.

* Newark, N. J.

Mr. Owen.

In the other two regions, the volcanic and the post-glacial formations, there is a certain amount of uncertainty in regard to the supply. In the volcanic formation with a slight cover, of course, there is no water unless one goes into the rock; and in either the trap, gneiss, or Atlantic formation, the supply is uncertain, erratic, and unreliable, and very rarely successful.

In the post-glacial formation, the drifts fill up the subterranean canals, as they may be called, and the terminal moraine depositions. In this case the water supply is fairly reliable within certain limits. In the speaker's experience there have been two or three curious propositions relating to this question. A certain city put in a water plant in a post-glacial deposition. When the wells were put down they overflowed. Pipes were driven down 80 or 100 ft., and left up in the air 10 or 15 ft., and still the wells overflowed. The pumping plant was started, and, of course, the overflow was gradually lowered. Incidental to that was the fact that, a little above the pumping plant, a man owned a very heavy spring which was totally stopped after a short period of pumping. He brought suit against the company, and, by a curious coincidence, the pumping plant broke down at a certain hour on a certain day, and a certain number of hours afterward, the spring ran afresh. That incident showed the underground capacity of that subterranean gulch, including the capacity of the underground storage.

That formation was almost all sand and gravel. The water-table in 3 years of pumping was lowered from an overflow of 6 or 8 ft. above ground to a ground-water flow of 22 ft. below the surface, showing that the company was pumping beyond the capacity of the delivery or the water flow of the country.

Take the other case, where there is a till flow, that is, in the glacial deposit termed the till, where the percolation is very slow. In certain cases under the speaker's care, the flow of the wells after a rainstorm was carefully timed, and the percolation, instead of being immediate, took about 2 days. In the case of one large well, where six or seven holes had been bored into the limestone, and where it was important to have supply enough for the examination, careful note was taken of the times when there was enough water and when the supply was deficient. Examination showed that, after a heavy rainstorm, it took 2 days for this flow through the till, that is, the drift deposit, to get through the rock and into the wells. This shows the extremely slow rate of percolation.

There are a great many interesting questions in the whole region about New York, and it is especially necessary to have known facts tabulated, as there has been a great deal of money wasted, especially in developing what is known as a mysterious underground supply.

In the case previously cited, the wells were put in for the city under the direction of a competent engineer, and the speaker remarked at the time that he was surprised at his being brave enough to put in such a plant for that community. The result has been that that city has been compelled to buy up land and territory after territory to provide a sufficient underground supply. Mr. Owen.

G. E. P. SMITH,* M. AM. SOC. C. E. (by letter).—The publication of this scholarly paper on investigations in the Owens Valley, and the studies and conclusions based thereon, must be welcomed by all hydraulic engineers of the arid West. In addition to its immediate or local value, the paper affords a clear and concise exposition of the general principles of ground-water storage, of recharge and loss, and of the extent to which ground-waters can be drawn on for municipal water supplies, irrigation, or other purposes. So far as the writer knows, it is the first comprehensive work of its kind, and the author has the distinction of being a true pioneer of the Profession. The paper itself marks a new era in the development of ground-water studies, an era in which scientific basis and logical methods supersede much idle speculation and many misconceptions. Mr. Smith.

Too much credit cannot be given to Mr. Lee for the thoroughness and application of system displayed in his investigations. These features are exemplified in the excellent analyses of rainfall distribution, of seepage losses, and of soil evaporation tests. Where arbitrary factors have been necessitated, they have been based on painstaking inspections and on rare good judgment. In such cases, also, the author was assisted by the extreme uniformity of natural conditions in the Owens Valley, especially of topography, valley fill, soil and vegetation. The application of system in the investigational work is reflected in its orderly presentation, and in the use of graphical methods, features which add greatly to the value of the paper.

The writer's purpose is not to criticize, but to broaden the foundation on which the deductions and generalizations of the paper are based, and to assist in the extension of their application to a wider range of climatic and geologic conditions.

Beginning in 1906, the Arizona Agricultural Experiment Station has carried on ground-water investigations under the writer's charge in several valleys of southern Arizona. These valleys vary greatly in the degree of aridity, from the grassy sub-arid Sulphur Spring Valley, which is comparable to Owens Valley, to the severely arid Lower Gila Valley. The chief determining factor is the altitude. The ground-water hydrology is exceedingly diverse in character, so that it is difficult to frame generalizations that are applicable to all the valleys.

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* Tucson, Ariz.

Mr. Smith.

The most detailed study has been that of the valley of the Rillito,* a tributary of the Santa Cruz, as shown in Fig. 13. There are striking similarities between it and Owens Valley. Both are desert valleys in which the surrounding drainage is toward a broad, flat area, or playa, and therefore they come within the meaning of the term "bolson" or "semi-bolson", as defined by Tolman.† The Pantano watershed spills its surplus water to the Rillito; the Rillito delivers a portion of its drainage to the Santa Cruz, and the Santa Cruz, on rare occasions, has a continuous discharge to the Gila River. Excluding the Pantano bolson with its 620 sq. miles, the total drainage area of the Rillito is 327 sq. miles, of which 56% is mountainous (granitic), 30% consists of dissected gravelly outwash slopes, and 14% is valley land. The area is nearly equal to that of the Independence Region, though the percentage of mountain area is higher. The lengths of the valleys are the same—25 miles—and both derive practically all their drainage from the right-hand side. The mountain rainfall, also, is equal in the two cases, varying from 10 or 12 in. in the foot-hills to 30 or 35 in. at the crests.

The distinctions are: the Rillito Valley lies at an elevation of about 2 400 ft.—considerably lower than Owens Valley—and consequently the temperature is higher and evaporation is greater; the mountainous area drained by the Rillito is on the windward side, and hence the rainfall is comparatively high for the region; the rainfall-altitude curve shows a continuous rise to the summit, at 9 000 ft.; and there is no perennial grass area, but, instead, the high mountains are forested with pine, and the lower slopes and valley floor are covered with a diversified desert vegetation, with mesophytic trees along the stream courses.

A survey of the water-table has shown that the river channel is not coincident with the ground-water trough; the ground-water on the south or left side has a component movement away from the river, especially during and after floods. The movement has been studied in lines of wells at right angles to the river, and the recharge which occurs during a flood season has been shown to progress away from the river as a true wave.‡ At no point has the water plane shown any response to direct precipitation, the rainfall penetrates only a few feet into the soil in the most favorable years, and it appears

* *Bulletin* No. 64, Arizona Agricultural Experiment Station, 1910.

† He states: "I therefore suggest that the word be used to cover the watershed of a centripetal drainage system, including all the area within the limits of the divides. The bolson may depart somewhat from a perfect topographical basin, for evaporation on a slope may prevent the development of a through drainage, and foster the centripetal variety. Those bolsons whose surface water in times of flood reaches some river thoroughfare some lower bolson, or the ocean direct, and consequently the playa portion * * * is poorly developed or lacking, may be called semi-bolsons."—*Journal of Geology*, XVII, No 2, p. 141, February-March, 1909.

‡ *Bulletin* No. 64, Arizona Agricultural Experiment Station, p. 184.

Mr. Smith.

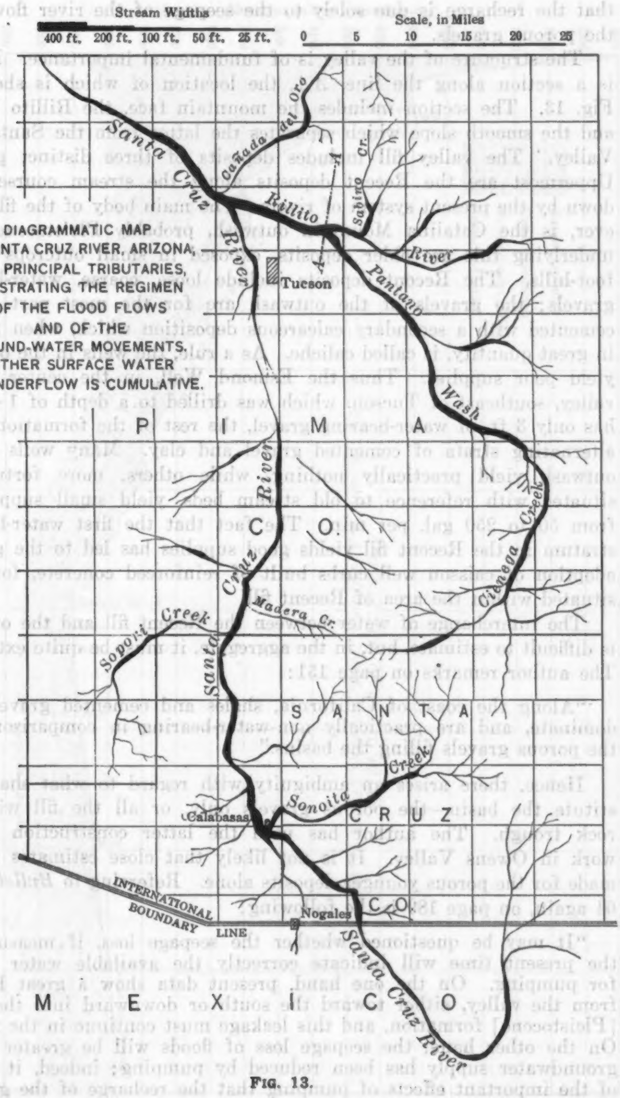


FIG. 13.

Mr. Smith. that the recharge is due solely to the seepage of the river flows into the porous gravels.

The structure of the valley is of fundamental importance. Fig. 14 is a section along the line, *AA*, the location of which is shown in Fig. 13. The section includes the mountain face, the Rillito Valley, and the smooth slope which separates the latter from the Santa Cruz Valley. The valley fill includes deposits of three distinct periods. Uppermost are the Recent deposits along the stream courses, laid down by the present system of rivers. The main body of the fill, however, is the Catalina Mountain outwash, probably Pleistocene; and, underlying this are older deposits, exposed in small outcrops in the foot-hills. The Recent deposits include loose, coarse, water-bearing gravels; the gravels of the outwash are for the most part tightly cemented with a secondary calcareous deposition which, when present in great quantity, is called caliche. As a rule, the wells in the outwash yield poor supplies. Thus the Esmond Well, in the center of the valley, southeast of Tucson, which was drilled to a depth of 1 480 ft., has only 3 ft. of water-bearing gravel, the rest of the formation being alternating strata of cemented gravel and clay. Many wells in the outwash yield practically nothing, while others, more fortunately situated with reference to old stream beds, yield small supplies of from 50 to 250 gal. per min. The fact that the first water-bearing stratum in the Recent fill yields good supplies has led to the general adoption of caisson well curbs built of reinforced concrete, for wells situated within the area of Recent fill.

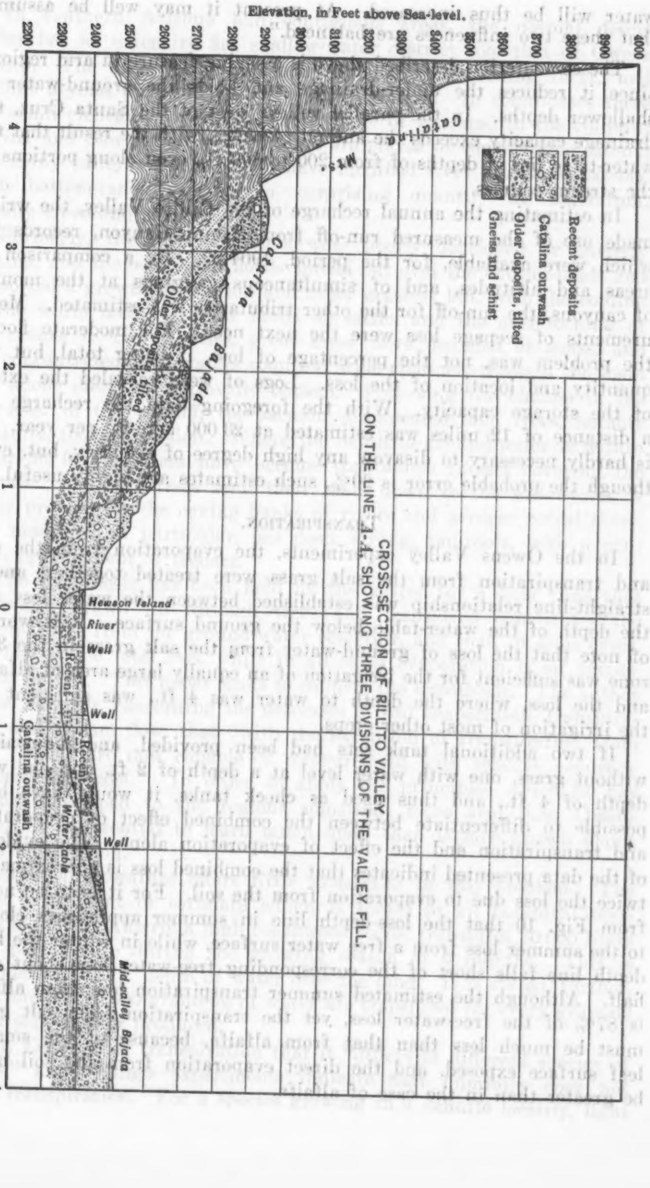
The interchange of water between the Recent fill and the outwash is difficult to estimate, but, in the aggregate, it must be quite extensive. The author remarks on page 151:

"Along the coast of California, shales and cemented gravels predominate, and are practically non-water-bearing in comparison with the porous gravels filling the basins."

Hence, there arises an ambiguity with regard to what shall constitute the basin—the porous gravels only, or all the fill within a rock trough. The author has used the latter construction in his work in Owens Valley. It is not likely that close estimates can be made for the porous younger deposits alone. Referring to *Bulletin* No. 64 again, on page 189 is the following:

"It may be questioned whether the seepage loss, if measured at the present time will indicate correctly the available water supply for pumping. On the one hand, present data show a great leakage from the valley, either toward the south or downward into the older [Pleistocene] formation, and this leakage must continue in the future. On the other hand, the seepage loss of floods will be greater if the groundwater supply has been reduced by pumping; indeed, it is one of the important effects of pumping that the recharge of the ground-

Mr. Smith.



CROSS-SECTION OF RILLITO VALLEY ON THE LINE 1-1 SHOWING THREE DIVISIONS OF THE VALLEY FILL.

FIG. 14.

Mr. Smith. water will be thus increased. At present it may well be assumed that these two influences are balanced."

The cementation described above is a saving feature in arid regions, since it reduces the under-drainage and holds the ground-water at shallower depths. In the parallel valleys west of the Santa Cruz, the drainage capacity exceeds the annual recharge, with the result that the water-table lies at depths of from 200 to 800 ft., even along portions of the stream courses.

In estimating the annual recharge of the Rillito Valley, the writer made use of the measured run-off from Sabino Canyon, records for which were available, for the period, 1904-09. By a comparison of areas and altitudes, and of simultaneous gaugings at the mouths of canyons, the run-off for the other tributaries was estimated. Measurements of seepage loss were the next need. For moderate floods, the problem was, not the percentage of loss, it being total, but the quantity and location of the loss. Logs of wells revealed the extent of the storage capacity. With the foregoing data the recharge for a distance of 12 miles was estimated at 23 000 acre-ft. per year. It is hardly necessary to disavow any high degree of accuracy, but, even though the probable error is 20%, such estimates are highly useful.

Transpiration.

In the Owens Valley experiments, the evaporation from the soil and transpiration from the salt grass were treated together, and a straight-line relationship was established between the water loss and the depth of the water-table below the ground surface. It is worthy of note that the loss of ground-water from the salt grass in the 3-ft. zone was sufficient for the irrigation of an equally large area of alfalfa, and the loss, where the depth to water was 4 ft., was sufficient for the irrigation of most other crops.

If two additional tank sets had been provided, and maintained without grass, one with water level at a depth of 2 ft. and one with depth of 4 ft., and thus used as check tanks, it would have been possible to differentiate between the combined effect of evaporation and transpiration and the effect of evaporation alone. The evidence of the data presented indicates that the combined loss is approximately twice the loss due to evaporation from the soil. For it is to be noted from Fig. 10 that the loss-depth line in summer approaches closely to the summer loss from a free water surface, while in winter the loss-depth line falls short of the corresponding free-water loss about one-half. Although the estimated summer transpiration loss from alfalfa is 87% of the free-water loss, yet the transpiration from salt grass must be much less than that from alfalfa, because of the smaller leaf surface exposed, and the direct evaporation from the soil must be greater than in the case of alfalfa.

In southern Arizona and contiguous areas, the normal desert valley has no extensive flat shallow-water district like that of Owens Valley. The water plane is usually at a depth exceeding 15 ft., and is thus beyond the influence of capillary action. In some cases there are small cienegas, covering an acre, more or less, and in the Sulphur Spring and San Simon Valleys there are playas of considerable extent; but many valleys of this region have modified river drainage systems, with bottom-lands carrying a surprising quantity of vegetation, usually trees and shrubs. Mr. Smith.

Formerly, the river channels were poorly developed or entirely lacking, and the occasional floods spread out over the bottom-lands and supported a growth of sacaton and other grasses; but, with the coming of the white man, with his herds of cattle, and the overstocking of the ranges in the Eighties, great areas were denuded of grass, and the concentrated flood-waters soon cut wide channels through the loam and adobe down to a gravelly bottom. Following the change in the character of the run-off, the sacaton disappeared, and trees, notably mesquite, took possession. Where the depth to ground-water is within 25 or 30 ft., the trees have become continuous forests covering the ground. That the trees send their roots down to the water-table is easily proven, for the caving banks of rivers and arroyas reveal them. The mesquite, in particular, has deep, strong, tap-roots, with a generous development of feeders.

The hypothesis has been held by the writer for a long time that the water drawn up through trees and transpired constitutes the principal loss from the ground-water reservoir, and that, in some cases, this loss is the total loss, while in all cases evaporation is an agency of less import. This paper tends to confirm the hypothesis. The author states, however, concerning the processes of soil evaporation, transpiration, and stream flow, that "with the exception of transpiration from trees", they "are now capable of measurement with relative accuracy at reasonable cost." In order to apply the principles which he has so ably developed to the regions somewhat more arid than Owens Valley, it is essential to learn much more than is now known about tree transpiration.

Botanical literature offers little assistance in this problem. The single experiment or estimate which is found in all botanical textbooks is that in Austria a high beechwood forest transpires 30 000 liters daily per hectare during the growing season of 6 months. This is equivalent to 22 in. depth of water over the land. It is stated, further, that from 250 to 500 grammes of water are transpired for every gramme of dry solid matter produced.

Mr. Lee says of transpiration that it "differs in different species of plants," and "King's experiments indicate that humidity does not affect transpiration. For a species growing in a definite locality, light

Mr. Smith. and available soil moisture are the controlling factors." These statements are surely open to question, as will be explained presently. It is probable that all plants tend toward the same rate of relative transpiration per unit of surface exposed, and the principle is well established that humidity is a potent factor of transpiration, as well as are light and soil moisture. The significance of these corrections lies in the fact that ground-water reservoirs are of chief importance in arid regions; and in such regions, with low humidity, brilliant light, and high rates of transpiration, even the xerophytic, or drouth-resistant, plants, under certain conditions, become most profligate in dissipating the only available water resources.

Recent researches of the Desert Botanical Laboratory of the Carnegie Institution have brought out some pertinent new principles of transpiration. The work of F. Shreve in the tropical mountain rain-forest of Jamaica has yielded the conclusions that humidity is a factor of much influence, and that transpiration is approximately proportional to evaporation. He says:*

"Although high humidities (90 to 95 per cent.) have been found to reduce the absolute rate of transpiration below its amount at relatively low humidities (55 to 71 per cent.), as is to be expected, the rate of relative transpiration continued to be of the same general order of magnitude at all humidities which are well above the minimal point for rain-forest plants."

Relative transpiration is a term used to define the ratio of transpiration to evaporation. Further, Shreve has studied data obtained by him in Jamaica in comparison with similar investigations carried on at Tucson, by B. E. Livingston on several desert ephemerals, and by Edith B. Shreve with the palo verde, *Parkinsonia,* which is rated as "a most successful desert tree". To quote again:†

"A general review of the data under comparison indicates that, in spite of minor differences, there is a greater uniformity among the relative transpiration maxima for the rain-forest and for the desert than might be expected. When such a uniformity is considered in the light of the fact that the evaporation is very many times greater in the Arizona desert than in the Jamaican rain-forest, it forces the conclusion that the transpiration-rates in the plants of the two regions must be roughly proportional to the evaporation-rates, else the relative transpiration-rates would not remain so nearly equal. In short, it is the desert plants in which the rate of transpiration is high and the rain-forest plants in which it is low, which is quite the reverse of the commonly accepted view."

G. F. Freeman, plant breeder of the Arizona Agricultural Experiment Station, has made simultaneous tests of transpiration on a peach tree and a creosote bush growing in close proximity. He found a very

* "Year Book No. 12," Carnegie Institution of Washington, 1913, p. 74.

† *Ibid.*, pp. 75-76.

slightly higher transpiration-rate for the peach tree per pound of green matter, but, when based on the area of leaf surface, the rate for the creosote bush was in excess.* The creosote bush is an extremely xerophytic plant, and forms the principal vegetation of the dryest slopes. Mr. Smith.

The establishment of the principle of equal relative transpiration-rates makes it possible to eliminate from many discussions factors such as light, air pressure, temperature, and humidity, and to base estimates of transpiration on the more easily measured evaporation from a free water surface. Also, the principle harmonizes the author's estimate of annual transpiration loss from alfalfa with King's estimate for clover in Wisconsin. The leaf characteristics of the two crops are very similar. The estimate for alfalfa is 41 in. depth of water transpired through the plants and 10 in. additional evaporated from the soil; for clover the estimate is 22.3 in. for both processes combined. The rates of evaporation in Owens Valley and in Wisconsin, however, are approximately as 2 to 1.† Since the rates for southern Arizona and for Wisconsin are as 2½ to 1, the inference can be drawn that in Arizona it requires about 56 in. for the proper irrigation of alfalfa.

Before condemning all desert vegetation indiscriminately, however, an ameliorating factor must be taken into consideration. Desert plants possess many queer habits by which they protect themselves from the tendency toward high transpiration-rates. On hot dry days they close their stomatal openings. In times of drouth they drop their leaves. The leaves are invariably small. The leaves of many species are covered with hairs. Some of the cacti are provided with water storage organs either in the roots or in the stems. Hence, plants pass successfully through seasons when the soil moisture around their roots becomes as low as 4 per cent. This power of self-protection is vital to the vegetation of the outwash slopes (usually but improperly called mesas), where the depth to the water level ranges sometimes as great as 600 ft. There is no evidence, however, that the power is exercised by trees growing on the bottom-lands, where the roots are bountifully supplied from below the ground-water table, or by alfalfa and other field crops, so long as they are well irrigated.

Adopting now the principle that transpiration varies as evaporation, soil moisture conditions being assumed to be uniform, the transpiration-rate for the beechwood forest previously quoted must be multiplied by 2½ to obtain the rate for arid regions. The result is 55 in. depth of water per annum, a quantity that probably represents fairly well the transpiration loss from the cottonwood trees which fringe the rivers for miles and are abundant usually on the upper courses of the tributaries. The loss from mesquite forest, on account of the

* Unpublished work.

† See new Evaporation Chart, "The Plant World," Vol. XIV, No. 9, 1911, p. 219.

Mr. Smith. smaller leaf expanse, is perhaps one-half as great as that from cottonwood trees.

As a corollary to the foregoing discussion, it is clear that the duty of water to be provided for in any locality is proportional to the evaporation-rate. On this basis, the high duty of water maintained in the San Bernardino Valley, in southern California, 7 or 8 acres per miner's inch, is no more creditable than the duty in Salt River Valley, Arizona, where for the last 3 years, the average duty of water, delivered, has been 5.4 acres per miner's inch. Following the over-use of water in many localities where it is abundant, some irrigationists now go to the other extreme, and advocate an impossibly high duty in the most desert valleys. Investigations relating to duty of water should be accompanied always by current observations of the evaporation rate, so that the results, when published, may be of more than local value. This precaution for making the investigations of wide interest has seldom been exercised.

A Principle of Ground-Water Hydrology.

That the ground-waters in arid valleys are not cumulative, is a principle which needs additional emphasis. Although there is a continuous movement of ground-water longitudinally in a valley, yet, in the main, the ground-water of one region does not get down into the next region. In this respect there is a close analogy between surface flows and underflows. Fig. 13 is a diagrammatic map of the Santa Cruz River and its principal tributaries. The widths of the lines are roughly proportional to the widths of the river channels, and indicate the regimen of the floods—the narrow torrents of the mountain canyons, and the spreading out over sandy beds and rapid absorption by percolation after leaving the canyon mouths. Normally, the river beds are dry. Many floods of the head-waters do not reach Calabasas, and few floods which pass Calabasas reach Tucson. The river is ever a dwindling stream, and, though draining greater areas, brings less water and smaller floods to the junctions with some of its tributaries than do those tributaries. Finally, at a point about 10 miles beyond the limit of the map, the channel is narrowed down to a width of 12 ft., and 2 miles farther on it entirely fades out. Likewise, the ground-water movements are represented by the same map. These movements are most active where the river beds are widest, where the recharging occurs; but at all points along the stream courses the moving ground-water is sustaining a loss, due mostly to transpiration, and the loss in any region is commensurate to the recharge in that region. The factors of recharge, loss, and forward movement are closely interrelated, though of varying importance in different localities within a region in their effects on the ground-water supply.

A logical sequence of these conditions is that extensive concentrated ground-water development is impossible. Vast areas of fertile land that could be reached easily by canals will ever remain desolate, but there are hundreds of small areas, usually stretching along the river courses, capable of reclamation by the utilization of ground-waters. The best of these small projects are to be found farther up stream. "The mountain water will not come down into the desert very far; we must go toward the mountain. The water must be used as near as possible to its source."* Mr. Smith.

Another sequence of the foregoing principle is that new diversions of ground-water in arid valleys are not so prejudicial to older rights as has been assumed oftentimes, and the doctrine of priority now applied to underflow streams of a definite character needs modification. In a discussion of the application of the doctrine to underflow gravity ditches,† the writer has suggested four points of limitation. They might well be applied to pumping operations also. The limitations relate to the burden of proof, to the extent of the injury, to co-operation in development work, and to the efficiency of the collecting agencies. Another consideration is that active interference between underflow ditches or between wells does not begin at once, but may require many months of lowering water plane, and, before the time has elapsed, one big flood may refill the gravels, whereupon all effects of the previous drafts on the ground-water supply are obliterated.

Estimates of Safe Yield.

The large items of the author's estimates carry much conviction as to their accuracy. Some of the small items, however, possibly need additional study.

1.—Percolation from Precipitation on Intermediate Mountain Slopes.—The Sierra slopes are said to be of unfissured granite covered by a mantle of loose rock. Table 16 gives the average annual rainfall as 12 to 20 in., derived mainly from the slow rains and snows of winter. Similar conditions exist on the slopes of the Catalina and Rincon Mountains, near Tucson, but observation indicates that most of the precipitation is absorbed by the porous overburden, wherein it supports an extensive desert vegetation. There is, of course, a slow creeping of the water downward at times, but very little of it reaches the valley. There are springs at the base of the slopes, but they are small, more suitable to be measured in cattle drinks than in second-feet. The run-off factors used for the intermediate mountain slopes of Owens Valley appear to be too high.

2.—The Upper Seeley Spring.—The location of this spring, at the north base of Charlies Butte, raises a question as to the derivation of

* *Proceedings*, 16th National Irrigation Congress, 1908, p. 206.

† 22d Annual Report, Arizona Agricultural Experiment Station, 1911, p. 570.

Mr. Smith. its water. If it is lateral flow from the outwash slopes, it is included rightly in the summary of ground-water losses, but if it is river underflow brought to the surface by the intervention of the butte—a hydrologic condition of common occurrence in the Southwest—then it should be omitted from consideration.

3.—It is stated that on the outskirts of the salt grass there is a strip of greasewood and bunch grass, that bordering this there is another zone of luxuriant sagebrush, and that the depth to water beneath these areas is from 8 to 20 ft. In the light of the foregoing discussion on transpiration, it is evident that the water loss from these bordering zones is of so much importance that an effort should be made to measure or estimate it.

4.—Another question of doubt to the writer is whether the underflow at Alabama Hills can be disregarded. The river bottoms here are 3 miles wide, and the rock trough is shown to be at least 832 ft. deep. The slope of the water-table is 8 ft. per mile. Although the valley fill on the Inyo Mountain side is composed of fine sand, yet on the opposite side there may be strata of good water-bearing gravels deposited by living rivers from the Sierra Range. A forward movement of the underflow of 6 in. a day throughout the section implies an item of loss of more than 20 sec-ft. This problem gives the writer more concern because in the arid Arizona valleys the bottom-lands do have usually some good water-bearing gravels. Thus, in the Santa Cruz "narrows", opposite Sentinel Hill, at Tucson, the underflow in the first water stratum must have been at least 10 sec-ft. before the recent development was made. The underflow loss can be measured or estimated by the Slichter method, and should be included in the balance sheet. Perhaps a fair estimate for Owens Valley is that the underflow gain at the north end of the Independence Region equals the underflow loss at the Alabama Hills.

In contrast with the author's orderly estimates are the crude methods which have found favor heretofore. A recent report on ground-water resources in an Arizona valley is based on absurd hypotheses, yet, on account of its authorship, the report ought to have been final and conclusive. The report first recites the water-shed area and rainfall, then applies Newell's run-off curves (which give values too high for arid regions), then deducts surface run-off, the quantity already applied to irrigation, and an allowance for evaporation from the dry river bed, and asserts that the remainder is underflow at the given cross-section of the valley. It ignores the largest factor of ground-water loss—transpiration—and assumes that the underflow is cumulative from the sources of the stream to the place under consideration. The magnitude of the underflow thus computed is excessively high, and is calculated to invite unwarranted expenditures in development.

On page 153 is the statement: Mr. Smith.

"There are * * * two possible methods of measuring the rate of recharge, either by determining the total percolation from various sources into the porous material of the basin, or by determining the ground-water losses."

The latter method is applicable in valleys where the loss areas are fairly compact and the vegetation fairly uniform; but, with a native vegetation of trees and shrubs, diverse in leaf expanse, in height, and in stand, and extending over irregular areas, there is little hope of approximating reliable estimates. On the other hand, as the recharge in very arid valleys is practically all from stream flows, the first method promises more accurate quantitative results. The essential features of the first method are an area survey and some gauging stations carefully selected, at the mouths of representative canyons, at the mouths of representative washes draining the outwash slopes, and near the outlet of the region. Fluctuations in wells, and water contour maps, present excellent qualitative studies, inasmuch as they reveal the direction of ground-water movements, the localities of active recharge, and the areas of loss. The first method has the disadvantage of requiring more attention to the residual losses in determining the safe yield than does the second.

The author has not discussed the probable variations in safe yield from year to year, though this is of great importance. Unfortunately, years of high rainfall, and again lean years, come in long series with great perversity. Rainfall records at Tucson show 6 years, from 1899 to 1904, with less than 10 in. of rain each year, followed by 5 good years with more than 10 in. each year, since which time there have been 4 years with rainfall below the average. The discharge from Sabino Canyon has varied from 2 900 to 46 100 acre-ft. per year. Ground-water levels respond more or less quickly to the variations in rainfall, and in some localities fluctuate over a wide range. Thus, along a section of Pinal Creek, the water-table has been as high as 12 ft. and as low as 85 ft. from the surface. The recent introduction of pump irrigation along the creek will increase the range of fluctuation.

Investigations of safe yield are most fortunate if made during a period when the rainfall is normal. In any case, the investigations should extend over several years, so as to eliminate the effects of storage from year to year. If data on the position of the water-table at many places can be secured during the year for which the estimates are made, the increase or decrease in the storage supply can be computed, and should appear as one item in the balance sheet.

Ground-water reservoirs of large capacity are better equalizers of the water supply than are surface reservoirs, but those of small capacity may be poor equalizers. In nearly all cases the highest utilization of ground-water requires the recognition of the principle of

Mr. Smith. fluctuating area under cultivation. Farmers are loath to admit the necessity for fluctuating area; the principle is not ideal, but in time it will be adopted by Courts and otherwise.

The author's summary of residual losses applies to many valleys perfectly. The outlet loss is not a loss to the State, however, inasmuch as it becomes available in the next valley region. What is termed the Artesian loss is, in many cases, not truly Artesian in character. This loss and the springs' loss are preventable by pumping operations. The loss by transpiration through trees is preventable, also, inasmuch as the feasible pumping lift is much greater than the limit of penetration of tree roots. Developments of the last 3 years in pumping machinery have doubled the economical limit for depth of pumping. Defining the efficiency of ground-water reservoirs as the ratio of the quantity recovered for irrigation to the total recharge, it is fair to anticipate, and to design works for, an exceedingly high efficiency, higher than many surface reservoirs, which lose from 3 to 10% through evaporation and an equal quantity through losses from a long supply canal.

Methods of increasing the safe yield require some variation from those proposed, in order to make them applicable to conditions in very arid valleys. Efforts to bring about greater absorption of flood-waters are not needed. The floods are wholly absorbed now, except for an occasional season of high rainfall, occurring perhaps once in 15 years. The efficiency of absorption through stream beds must be higher than that of absorption on an exposed slope. The only promising method is the elimination of transpiration through the denudation of the bottom-lands. The native forests and scattered trees should be removed, and, so far as the water supply permits, replaced by vegetation that is useful to man.

In conclusion, the investigations in Owens Valley are timely, for the extensive development of ground-waters in the Southwest points already to the over-draft, and in some places to the exhaustion, of the supply within a few years. The Engineering Profession, and not the Courts of Law, should take the prominent part in the final adjustment of pumping operations to the limiting physical conditions.

Mr. Meinzer. O. E. MEINZER, ESQ.* (by letter).—Mr. Lee's investigation in Owens Valley is a valuable contribution to the study of ground-water. The demand for irrigation supplies and the increasing availability of ground-water because of improved irrigation and cultural methods and decreased pumping costs have created a need for information as to the magnitude of ground-water supplies, the question being primarily not as to the quantity stored in the earth but as to the annual recharge or safe yield. Any contribution to the methods for estimating the

* In Charge, Ground-Water Div., Water Resources Branch, U. S. Geological Survey, Washington, D. C.

annual supplies of ground-water, therefore, is especially valuable at this time. Four principal methods or groups of methods, which may be called the percolation, underflow, water-level, and evaporation methods, have been used. The first consists in estimating the quantity of water that percolates into an underground reservoir from streams or other surface sources; the second in measuring the flow of ground-water at selected cross-sections, being similar in principle to the gauging of surface streams; the third in observing fluctuations in the water-table, which represent filling or emptying of the underground reservoir; and the fourth in measuring the discharge of ground-water through evaporation from soil and plants. All these methods are laborious and difficult to apply, and none of them can be expected to produce precise results, but they are valuable, nevertheless, because they give some tangible basis for estimating ground-water supplies. In the Owens Valley investigation, Mr. Lee used both percolation and evaporation methods, his distinctive contribution consisting in developing the latter method and placing it on a quantitative basis. Mr. Meinzer.

The evaporation method gives promise of extensive applicability because a large number of the ground-water reservoirs that will be developed for irrigation discharge wholly or chiefly by evaporation. Débris-filled basins are the most characteristic features of the United States west of the Rocky Mountains. They are of two types: Those which discharge ground-water through springs and evaporation areas, as described by Mr. Lee, and those which do not. In the geologic literature dealing with these basins this fundamental distinction is generally ignored, and the characteristics of the evaporation areas are commonly accounted for by the wholly inadequate explanation of evaporation of surface waters. The process of ground-water evaporation, however, has been clearly stated by F. H. Newell,* M. Am. Soc. C. E., in one of the earliest papers on water resources published by the United States Geological Survey, and in recent ground-water investigations the significance of evaporation areas has come to be clearly recognized. Mr. Lee has furnished experimental data showing that these areas are quantitatively important in discharging ground-water.

Work in numerous débris-filled basins has shown that it is entirely feasible to ascertain from surface indications whether or not a basin is discharging ground-water by evaporation. The evaporation areas in some of the basins have been mapped with nearly the same accuracy that is possible in mapping geologic formations. The three criteria, all of which are suggested in the paper, are (1) moisture of the soil; (2) soluble salts at the surface; and (3) native plants that feed on ground-water. Experience is necessary, of course, for a proper application of these criteria, but they are trustworthy when rightly used. Moreover, they can be tested at any time by making a shallow boring,

* "Water Supply for Irrigation," U. S. Geol. Survey, 13th Annual Rept., 1893. Pt. 3, p. 29.

Mr. Meinzer. for ground-water evaporation takes place only where the water-table is near the surface. In a recent ground-water survey of the Big Smoky Valley, Nevada, the relations between the vegetation zones and the depth to the water-table were found to be very similar to those stated by Mr. Lee (page 198), but the plant species that serve as indicators of ground-water are not the same in different parts of the West. The mapping of evaporation areas is important, not only because these areas make possible estimates of the annual supplies, but also because they reveal the base level of the ground-water surface, and thus make possible a forecast of the depth to water in other parts of the basins in which they occur.

Mr. Lee's first conclusion (page 149) is open to criticism in being too general. On the basis of his investigation in the Owens Valley, he concludes that the underground reservoirs of California and the Southwest are water-tight rock basins. Many of the débris-filled basins of the Southwest, however, even those comparatively well enclosed by mountains, have no ground-water discharge through springs, evaporation, or transpiration, and it must be assumed that the supplies which they undoubtedly receive are disposed of entirely through rock absorption or underground channels of escape. Some of the reservoirs which discharge water by springs, soil evaporation, and transpiration, no doubt also suffer losses through rock absorption or leakage. In this respect each basin forms a separate problem, the amount of underground loss depending on the stratigraphy and structure as well as the topography. It should be noted that such loss, if heavy, will make the estimates obtained by the percolation method too large and may make those obtained by the evaporation method too small. One of the sources of the public supply of Goldfield, Nev., has consisted of wells in a closed débris-filled basin which has no discharge by evaporation. Such a basin will yield some water even though it has no natural discharge through springs, evaporation, and transpiration, but its yield will be less than the quantity that percolates from surface sources to the water-table.

The writer was disappointed in not finding in Mr. Lee's paper a more definite discussion of the probable percentage of accuracy of his results, as such a discussion would have added greatly to the value of the paper. Mr. Lee's analyses both of recharge and of loss are excellent, but in order to reach quantitative conclusions he was obliged to make numerous assumptions in respect to both. Although these assumptions were, the writer believes, made carefully and with good judgment, they must have introduced considerable errors into the computations. As assumptions enter into both estimates, neither one can be regarded as a check on the other. The fact that the two estimates are of the same magnitude justifies added confidence in the general results, but the fact that they differ by less than 4% does not indicate, of course,

that the percentage of error is within 4%, and can hardly be considered a confirmation of the reliability of the data and the correctness of the assumptions in either computation (page 212). Mr. Meinzer.

The four sources of percolation are given as (1) direct precipitation; (2) stream flow; (3) irrigation; and (4) flood-water in the valley floor. It is assumed that the high mountain areas shed all precipitation except that which is lost by evaporation; that on the more elevated parts of the intermediate mountain slope 75% of the precipitation, and on the lower parts 70, 65, and 60%, respectively, join the underground supply; that the contributions to the underground supply on the outwash slopes range, according to zones, from 0 to 60%, making an average of 16%; and that the contribution on the valley floor is 4 sec-ft. All these assumptions are based on careful, general observations, but are of course only approximations and subject to large errors. Moreover, assumptions are involved as to the areas belonging to the various zones and the amount of precipitation in each (page 206). Several assumptions are also made in the estimate of recharge from stream channels, in the conclusion that 50% of the irrigation water is added to the underground supply, and in the estimate of the contributions made by the flood-waters.

The writer agrees with Mr. Lee that the discharge from the underground reservoir can be determined more accurately than the percolation into it, but the methods of estimating discharge also involve a number of elements of uncertainty. If the writer understands rightly the author's discussion of the discharge from irrigated land, the 50%, or 18 sec-ft., of irrigation water discharged by evaporation and transpiration (page 203) is the portion that was not added to the underground supply (page 212) and, therefore, should not be included with the discharge from this supply.

The largest element in ground-water discharge is that of evaporation and transpiration from uncultivated land. It is on this question that Mr. Lee has made his most important contribution, and that part of his paper dealing with this phase of the subject deserves, therefore, the most critical consideration. Some of the factors that enter into this estimate were determined by experiment and others by field survey. Experimental errors were involved in the difference between natural and artificially packed soils, in the difference between the vegetation in Nature and in soil tanks (pages 186, 187, 192, and 193), in uncertainties as to the actual water-level in the tanks (pages 183 and 184), and in the unavoidable fluctuations in the water-levels in each tank (Tables 9 to 14, inclusive). These experimental errors are represented in part only in the diagrams in Fig. 10. In the summer diagram, which is the more important one, the results from Tanks Nos. 5, 6, and 7 fall nearly on the curve used by Mr. Lee in his calculations, the result from Tank No. 4 being more than 20%

Mr. Meinzer. too low, that of Tank No. 3 more than 20% too high, and that of Tank No. 2 nearly 40% too low. It should be noted, however, that although these results involve large experimental errors, they corroborate each other in a general way and are of great value in furnishing reliable data on the general magnitude of one of the most important processes in ground-water circulation.

In applying the experimental data to the basin under investigation, inaccuracies were involved in the sizes of the areas having specified depths to the water-table, in the fluctuations of the water-table, in difference in the range and rate of capillary rise due to difference in the character of the soil, and in difference in the density of vegetation and kinds of plants that draw water from the underground reservoir. An error was also involved in the fact that observations covering only 1, 2, or 3 years did not give average evaporation conditions, just as precipitation and stream-gauging data for a period of the same length do not afford reliable averages. With 142 observation wells on the valley floor (page 197), the error as to the water-table cannot have been large, yet the rate of discharge varies so greatly with small changes in depth of the water-table that even slight inaccuracies in determining one produce appreciable errors in calculating the other. Although the valley floor in that part of Owens Valley investigated by Mr. Lee has relatively uniform conditions, there is luxuriant salt grass in some parts and an entire absence of vegetation in others (page 161), whereas the experiments did not cover these different conditions. Moreover, no account was taken of the zone having depths to the water-table between 8 and 12 ft., although the greasewood and rabbit brush in this zone probably draw water from the underground reservoir (page 198).

The writer agrees with Mr. Lee that the evaporation method is the most feasible one for estimating ground-water recharge in many of the débris-filled basins, especially where there are as yet few developments, but its application is far from being a simple matter. The data which he obtained in Owens Valley have value in making rough estimates of annual recharge in valleys in which the evaporation areas are mapped and reliable observations are made as to the character of the soil and vegetation and the distance to the water-table in the different zones of such areas, but to assure any considerable degree of accuracy for most valleys it will be necessary not only to sink a large number of observation wells and to keep them under observation for one or more years, as was done in Owens Valley, but also to obtain a great deal more information as to the rate of discharge under various conditions of soil and vegetation. The conditions in the evaporation areas of most valleys are far from uniform, the soil ranging from dense clay to coarse sand or gravel, and the vegetation embracing a number of diverse species and being entirely absent over large tracts.

The writer wishes to urge the importance of further investigations along the lines suggested. The lowest parts of many of the closed basins are underlaid by clay cores destitute of vegetation but surrounded by zones of less dense soil in which evaporation and transpiration are active. The dissemination of the soluble salts instead of their concentration at the surface and other conditions lead him to believe that on these clay cores the ground-water discharge is sluggish, but definite tests are needed as to the quantity of discharge and its relation to the distribution of the soluble salts. Among the common native plants (besides the salt grasses) which apparently discharge ground-water, are samphire (*Spirostachys occidentalis*), iodine weed (*Suaeda*), alkaline sacaton (*Sporobolus airoides*), certain species of salt bush (*Atriplex*), big greasewood (*Sarcobatus vermiculatus*), rabbit brush (*Chrysothamnus graveolens*), buffalo berry bush (*Shepherdia*), and mesquite (*Prosopis*). Mesquite does not thrive in the shallow-water areas where the soil is dense and alkaline, but is often dominant in a zone of moderate depth to water surrounding a shallow-water area. It is important to know whether the mesquite actually feeds on ground-water, and if so, from what depth and at what rate it is able to lift this water to the surface. **Mr. Meinzer.**

Mr. Lee's final conclusions (page 218) sum up admirably the esssential factors in the problem of safe yield. As it is not generally practicable to draw any large part of the ground-water of one segment of a valley to another, a proper distribution of wells is necessary in order to reduce the residual losses to the lowest possible quantity. Overdraft with serious lowering of the water-table may occur in certain segments while evaporation of ground-water contributed by other segments is still in progress on the lowest lands. Such loss can be prevented only by increased withdrawals in the undeveloped segments, and if these segments have little or no land that can be profitably irrigated, the loss may be unavoidable.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—Underground water supplies, as well as surface supplies, depend on the rainfall, the catchment area, and the available storage, with its accompanying losses by overflow, leakage, and evaporation; but the problem for the engineer is more difficult in the case of underground supplies, as the true limits of the catchment area, the storage capacity, and the probable amount of the losses mentioned can only be determined approximately after a pretty thorough examination of the sub-surface conditions, that is, the configuration of the permeable strata, their impermeable confines, and their physical characteristics—permeability, etc. In the majority of cases much of this information is inaccessible, and more or less dependence is placed on such collateral evidence as the yield of neighboring wells and the results obtained by sinking test wells. **Mr. Allen.**

* New York City.

Mr. Allen. The limitations imposed by enclosing impervious formations and by sub-surface evaporation are clearly and interestingly presented by the author, and their importance is shown under the conditions discussed. In most well developments, however, evaporation is of minor significance. This is not only on account of greater humidity, but because most supplies percolate through an unenclosed stratum for perhaps many miles and because evaporation from this stratum is usually prevented by the superposition of one or more impervious strata.

In practice the available yield is subject to further limitations. The water-bearing stratum may consist of an impervious rock containing crevices or fissures due to movements in the earth's crust, or water-courses caused by the solvent action of the water itself. The latter are of frequent occurrence in limestone and chalk formations and the former in sandstones and the denser igneous rocks. Fissures occur more frequently near the surface than at great depths, and, as the cost of drilling increases with the depth, deep borings in search of water-bearing fissures are not often profitable. The same money can be spent to better advantage in making several test borings at moderate depth. Although the results of such borings are uncertain, supplies, when obtained from fissures, are often abundant and the wells free from the clogging so often experienced with sand. The yield of a well in sand is directly limited by the porosity of the latter and the available head. In fine sands there is a large loss of head due to percolation near the strainer in order to maintain the necessary flow. This difficulty may be overcome by increasing the length of the strainer, if this does not exceed the thickness of the water-bearing stratum, but this is at the expense of a greater first cost as well as an increase in lift due to the fineness of the material. In such cases the loss of head is often reduced by removing the sand about the strainer and filling the pocket with gravel, or else by substituting a larger number of driven well points from 2 to 3 in. in diameter for the large (6 to 12-in.) casing and strainer.

In those cases where the water occurs in a stratum of fine sand of small thickness, the difficulty is further increased. The writer tested a stratum of this kind several years ago with the view of developing a supply of some 5 000 000 gal. daily for a southern city. Water was found in apparent abundance throughout a large area at a moderate depth, the land was low, covered with forest, not far from a stream of considerable size, and on it were several large springs. On sinking numerous test wells on a line about 4 miles long, however, it was shown that the water-bearing stratum, besides being of a fine sand, was so thin that the cost of developing the desired supply would be prohibitive.

Water supplies found in deep deposits of sand or drift, especially those derived from distant and extensive catchment areas, such as

exist south of the great terminal moraine on the south side of Long Island and on the Atlantic Coastal Plain from Sandy Hook to Florida, are most favorable for development. Well-known examples are the 1 400-ft. Ponce de Leon well at St. Augustine, said to furnish 10 000 000 gal. daily, and the 1 970-ft. well at Charleston, furnishing 1 250 000 gal. daily. On the other hand, a well was bored in 1900-01 at Atlantic City, N. J., to a depth of 2 306 ft. (below the floor of Young's Pier), and although a good supply was found between depths of 780 and 860 ft., in a stratum of sand tapped by numerous wells in the vicinity, the supply sought at the greater depth was not realized, and the well, then the deepest but one along the Atlantic Coast, was abandoned. Mr. Allen.

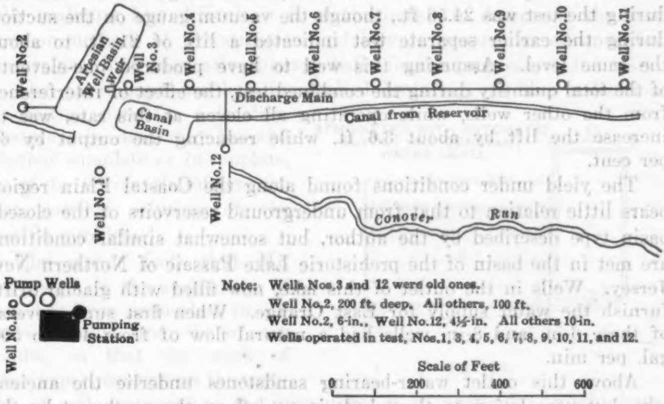


FIG. 15.

About 5 miles inland, the Water Department had a small collecting basin near its pumping station (Fig. 15), in which were driven a number of 4-in. tubes tapping a water-bearing stratum 24 ft. below. Near the basin was a 10-in. well (No. 3) extending to a depth of about 100 ft. to another water-bearing stratum which it was proposed to develop as an additional supply to the extent of 3 000 000 or 4 000 000 gal. daily. The capacity of this well was first tested by erecting over it an old 1 000 000-gal. Worthington pump, which the Department had, and connecting it with a steam line to the boiler plant at the station. During a 6-hour test with a 19-in. vacuum on the suction, the delivery was practically uniform at a rate of 840 000 gal. per day, and as no downward flow was observed in the small tubes in the basin, it was concluded that there was no connection between these two strata. On the strength of this test, ten 10-in. wells, 125 ft. apart, were sunk to a depth of about 100 ft. and a 6-in. well (No. 2) to a still deeper

Mr. Allen. stratum, 200 ft. below the surface. Besides these there were two old wells—the 10-in. well tested with the steam pump and a 4½-in. well (No. 12), both of which were carried to the 100-ft. stratum. The deeper 6-in. well flowed freely at a rate of 66 000 gal. per day and, by use of the air lift, at a rate of 400 000 gal. per day, or about 280 gal. per min. The 10-in. wells on short separate tests produced from 150 to 400 gal. per min. Ten of the 10-in. wells (No. 1 and Nos. 3-11) and the 4½-in. well (No. 12) were then connected with the compressor and given a 6-hour test. These wells together delivered 846 879 gal., or at a rate of 3 705 492 gal. per day. This was an average of 336 866 gal. per well, or 60% less than the delivery from Well No. 3 when operated alone. Moreover, the average lift in this well during the test was 24.56 ft., though the vacuum gauge on the suction during the earlier separate test indicated a lift of 21 ft. to about the same level. Assuming this well to have produced one-eleventh of the total quantity during the combined test, the effect of interference from the other wells, while operating all eleven at this rate, was to increase the lift by about 3.6 ft. while reducing the output by 60 per cent.

The yield under conditions found along the Coastal Plain region bears little relation to that from underground reservoirs of the closed-basin type described by the author, but somewhat similar conditions are met in the basin of the prehistoric Lake Passaic of Northern New Jersey. Wells in the outlet of this lake, now filled with glacial drift, furnish the water supply for East Orange. When first sunk, several of these 6-in. and 8-in. wells had a natural flow of from 400 to 500 gal. per min.

Above this outlet water-bearing sandstones underlie the ancient lake, but percolation to these beds is cut off on the northwest by the trap dikes forming the Orange or Wachung Mountains and on the southeast by the Palisades of the Hudson. Assuming a general southerly flow, percolation from a distance, therefore, is limited to that from the northeast, the limit in this direction, it is believed, being unknown.

These sandstones, nevertheless, furnish a large number of excellent well supplies in the vicinity of Newark and Paterson, chiefly by means of their numerous fissures.*

Data concerning a large number of these wells, collected by the writer a few years ago, indicate a great variation in the yield. They are commonly from 100 to 500 ft. in depth, and 6 or 8 in. in diameter, and generally furnish from 20 to 200 gal. per min. each, although failures are not infrequent, and several produce as much as 500 gal. per min.

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* Report, State Geologist of New Jersey, 1903, p. 79; also Water Supply and Irrigation Papers, U. S. Geological Survey, No. 114, p. 96.

Mr. Horton.

Robert E. Horton,* M. Am. Soc. C. E. (by letter).—This paper bears abundant evidence of being the work of an accomplished hydrologist, and specially commends itself to the writer because it represents the combined application of studies of the rainfall and run-off relations of the basin itself with laboratory experiments on evaporation losses.

For many purposes in hydrologic work, laboratory experiments are capable of yielding very instructive results because the problems in Nature are often so complex as to make it difficult to separate effects produced by one cause from those produced by a combination of causes. After studying the completed results, various ways suggest themselves by which the experimental data might have been improved. The writer, however, refrains from criticism in this regard, fully realizing how difficult it is at the outset to determine the best lines or methods of investigation and how to prepare in advance for conditions which may arise unexpectedly in the progress of the work.

The paramount problem in applied hydrology is the utilization of existing data for a locality, whatever the data may be, and whether complete or incomplete, so as to derive therefrom the best possible solution of the specific problem.

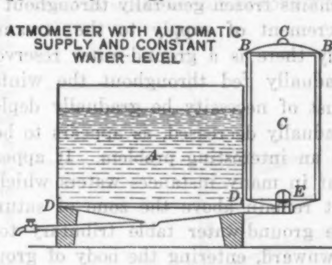

Fig. 16.

The writer feels that Mr. Lee's work in the acquisition of data has been somewhat in advance of his analysis of the results, in that the work of other experimenters on the question of soil evaporation, and transpiration in particular, might have been analyzed and to some extent utilized to advantage in this case. For example, it would seem that the difficulties experienced in obtaining constant ground-water table in the soil evaporameters should have been foreseen. Quite similar experiments were carried out successfully by Ebermayer many years ago, using the apparatus shown in Fig. 16, in which the lower surface of the soil prism is maintained constantly in contact with the water surface by an automatic air valve in the reservoir, *C*. The uplifting of water from the soil prism is entirely the result of capillary action.†

The writer has not had opportunity to examine Water Supply Paper No. 294, which presumably contains detailed results of the observations of precipitation and run-off. It would have added to the value and interest of the paper if Mr. Lee had included tables

* Albany, N. Y.

† A full description of Ebermayer's experiments and results is given in a paper by the writer in *The Michigan Engineer*, 1913-14, pp. 66-89.

Mr. Horton. showing the monthly precipitation and run-off of the different streams, because it is unusual to find records of the attendant conditions available in conjunction with stream flow data, as is the case here.

Mr. Lee states that the flow of the mountain gulch streams tributary to Owens Valley is practically constant throughout the frozen season. This statement is only relatively true. From the data relating to these streams appearing in Water Supply Paper No. 300, their yield varies 100% or even more, there being generally a gradual decrease in flow from the beginning to the end of the frozen season. Attention is called to this point principally because a fundamental principle of the regimen of streams is commonly overlooked, namely, that a stream or spring cannot yield a constant outflow throughout any considerable period of time unless there are simultaneous additions to the available water supply. In the case of the mountain streams in question, the winter precipitation occurs as snow and remains frozen generally throughout that period, thus eliminating any increment of supply to the streams from surface water. Presumably there is a ground-water reservoir from which these streams are gradually fed throughout the winter. This ground-water reservoir must of necessity be gradually depleted and the flow of the streams gradually decreased, as appears to be the case. This, however, brings up an interesting problem. It appears possible, and indeed probable, that in many instances waters which have previously entered the soil but remain above the zone of saturation—or, in other words, above the ground-water table tributary to the stream—gradually percolate downward, entering the body of ground-water and aiding to maintain a constant supply through long periods of drought or during the winter when surface supply and infiltration are shut off. If the rate of addition to the ground-water body is greater than the initial rate of outflow therefrom, the ground-water table will gradually rise to such a height that the outflow will become equal to the inflow from percolation, and then, as long as the downward percolation remains substantially constant, the yield of the stream also will remain constant.

Mr. Lee's paper brings out forcibly an important hydrologic fact often overlooked, namely, that very substantial yields of water may often be obtained permanently from undrained depressions or closed basins where naturally the entire precipitation is lost through evaporation. To accomplish this it is only necessary to reduce the evaporation as much as possible below the inflow from precipitation. This, as pointed out in the paper, will naturally result if the ground-water table is drawn down permanently below the limit of capillary uplift and evaporation from the soil, and below the limit of absorption by plant roots.

Mr. Horton.

Mr. Lee describes the typical geological construction of a mountain fault basin as the result of the product of faulting accompanied by the uptilting of a crustal block from one side of the line of fracture. Fault valleys of this type are not uncommon in the East. One of these, in the eastern slope of the Helderbergs, not far from Albany, is known to the writer. In this case a section of the valley is apparently somewhat as shown in Fig. 17. It appears to the writer probable that in this case, and no doubt frequently in other cases, there are more or less débris-filled spaces, *A* and *B*, between the ends of the tilted blocks and the fault face. As a result, there are likely to be subterranean outlets from the valley through the spaces, *A* and *B*, at greater or less depths below the junction of the valley surface rock with the fault face at *C*. In the vicinity referred to, frequent illustrations of tilting and overthrust of underlying strata can be seen. Evidence of subterranean channels along the fault line beneath the rock floor of the valley appears from the fact that the only visible outlet of drainage from the valley is through a very large and permanent spring which comes to the surface along a continuation of the fault line at a distance of about a mile from the lower end of the valley and at the point where the fault line crosses the valley of a large stream.

FAULT IN MOUNTAIN VALLEY FORMATION

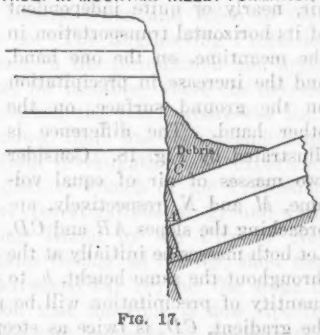

FIG. 17.

This instance, and the frequent occurrence of undrained depressions in the East, suggests to the writer the fact that the underlying principles of hydrology are very general, and the conditions widely distributed throughout the earth. Unfortunately, the range of conditions is not as well recognized as it should be, and it is probably due to this fact, rather than to differences in the classes of conditions to be met, as classes, that there have been so many wild and erroneous estimates of the available yield of water supplies. In view of the limited experiments and studies which have been made along scientific lines such as those carried out, for example, by Mr. Lee, for the solution of specific problems, it becomes an increasing matter of wonderment to the writer that serious mistakes and miscalculations in the available water supply for municipalities or for power or other purposes have not more often been made.

It is often the case that the success or failure of a hydraulic project depends primarily on the water supply, yet, as a rule, much less attention is given to the scientific determination of this element

Mr. Horton. than to many much less important structural elements. The writer is pleased to see in the present instance the hydrologic side of the question at issue given apparently its due share of attention.

The writer feels obliged to take exception to Mr. Lee's statement that the transpiration from plants is independent of the humidity. The elegant experiments along this line by Ganung, in connection with which graphic records of transpiration and of the accompanying meteorological elements, light, humidity, and temperature, were obtained, illustrate quite conclusively, it would seem, that there is a fairly definite relation between transpiration rate and humidity.*

Mr. Lee has proved that the increase in rainfall on mountain slopes is more nearly proportional to the slope gradient than to the absolute humidity. It seems necessary to call attention to the distinction between the increase in atmospheric precipitation which, as shown by the classic researches of Pockels, is directly the result of dynamic cooling and consequently is a function of the elevation of a mass of air, nearly or quite independent of its horizontal transportation in the meantime, on the one hand, and the increase in precipitation on the ground surface, on the other hand. The difference is illustrated by Fig. 18. Consider two masses of air of equal volume, M and N, respectively, approaching the slopes AB and CD.

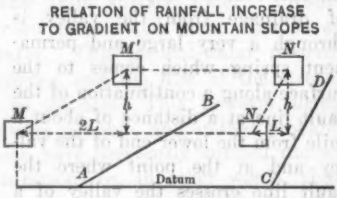

FIG. 18.

Let both masses be initially at the same height, and let both be elevated throughout the same height, h, to the positions, M' and N'. The same quantity of precipitation will be produced from each mass of air. If the gradient, CD, is twice as steep as the gradient, AB, the resulting precipitation on the flat slope, AB, will be distributed over twice as great an area as that on the slope, CD, as is evident from the diagram. Thus the apparent increase in precipitation will be proportional to the slope gradient.

The peculiar condition existing in the case of triangular mountain drainage areas, whereby the mean precipitation occurs at the center of area of the drainage basin, illustrates the importance of weighting rainfall records, taken at different portions of a drainage basin, in proportion to the relative part of the total area which each one represents. This practice has been followed by the writer for a number of years in estimating mean rainfall on drainage areas. The necessity for the use of such a process arises partly from the fact that rainfall stations are most commonly located in stream valleys. In the case of triangular basins, similar to those tributary to Owens Valley, but

* "The Living Plant," by W. F. Ganung, p. 204.

for the extreme condition of zero precipitation at the mouth of the stream, the average of a line of equidistant rainfall stations located along the stream from its source to its mouth, would give an apparent mean precipitation for the area only five-sixths as great as the true mean precipitation, or the error would be $16\frac{2}{3}\%$ of the true mean. Mr. Horton.

CHARLES H. LEE,* ASSOC. M. AM. SOC. C. E. (by letter).—Realizing the somewhat local character of his paper, the writer has been highly gratified to find that it has brought forth discussion based on such widely divergent as well as similar experience. He especially appreciates the broad and constructive contributions of Messrs. Smith and Meinzer, whose wide experience with ground-water problems qualifies them to speak with authority. Mr. Lee.

The writer heartily agrees with Mr. Owen that there is a great need in all sections of the United States for definite scientific knowledge regarding the principles which should govern ground-water development. He would not, however, place such knowledge in the realm of the unattainable, as both Messrs. Owen and Allen in their opening paragraphs seem inclined to do. The Engineering Profession has already made considerable progress in the formulation and application of such principles. In Europe, for instance, where the demand for municipal water supplies has reached the point where the limit of every available source must be known, there has been accumulated such a fund of knowledge derived from investigation and experience that the subject is recognized as a distinct branch of the Profession. In the United States the exhaustive investigations of ground-water supply, such as that made on Long Island by the City of New York and the work of the United States Geological Survey, show that in America, also, the subject has advanced far beyond the stage of conjecture. It is the writer's opinion that the time is now ripe for the formulation of general principles and methods of investigation and development of ground-water among American engineers, and it was with this in view that his paper was written. It is to be hoped that similar papers will be presented by members of the Society who have had opportunity to study other types of ground-water occurrence, in order that a more complete presentation of the subject in its most recent development may become available to the Profession.

The writer was much interested in the statements of both Messrs. Smith and Meinzer that the term "water-tight" as applied to ground-water reservoirs is a relative one, and that, in many arid States, it does not apply at all. Their experience and the writer's more recent observations outside of California are in accord in this matter, and the first conclusion of the paper (page 149) should be modified accordingly. In all the California basins which have come under the writer's

* Los Angeles, Cal.

Mr. Lee. observation, however, the term can be used for all practical purposes. These basins occur either in granite or in tertiary sandstones and shales, and, as the fill is recent, the difficulty encountered by Mr. Smith in defining the basin seldom arises.

Both Messrs. Meinzer and Smith have commented on transpiration losses, and the latter has contributed the first really useful statement of basic principles which the writer has seen. It is to be hoped that further work will be done to confirm the principle of equal relative rates of transpiration, and that quantitative determination be made of the ratio between evaporation and transpiration. In all such experiments care should be taken to measure evaporation under some standard condition, for, as is well known, the results derived from pans in large bodies of water, in wet soil, in dry soil, and in air, differ widely, and in the same environment differ with atmospheric exposure.

In this connection the writer wishes to call attention to the fact that Mr. Smith's conclusion (page 226) that the combined loss by evaporation and transpiration from salt-grass sod is twice that from damp soil alone is not justified. The rate of water evaporation from the deep tank in the soil far exceeded that from soil during the winter, for the reason that the soil was kept cold by evaporation and cold air temperature, while the water was free to circulate and received warmth from the deeper soil. Experiments (which the writer carried on but did not publish) indicate that the annual evaporation from saturated base soil is about 93% of that from the water surface in the deep tank. The combined transpiration and soil evaporation for similar conditions was 115 per cent.

With these general observations the writer will pass on to detailed comments on the discussion of the paper. Mr. Smith's paragraphs, grouped under the head of "Estimates of Safe Yield" (pages 231-234), will be taken up first.

1. The conditions on the slopes of the Catalina and Rincon Mountains, described by Mr. Smith, are not as similar to those of the east slope of the Sierra Nevada as would appear at first glance. Table 3 shows that the lowest elevation of the Sierra slopes is 6 500 ft., the average 7 500 to 8 000 ft., and the maximum elevation 12 000 ft. The similar slopes back of Tucson extend from an elevation of 3 000 to about 6 000 ft. The Sierra slopes do not support a luxuriant desert vegetation nor forest growth, the latter, especially, being spotted and sparse. The precipitation on the Sierra slopes is all in the form of snow, which melts and sinks into the porous mantle before the growing season commences. On the intermediate slopes of the Catalina Mountains, however, the precipitation occurs largely in severe summer storms, from which the run-off is rapid and the percolation is immediately available to vegetation. These differences are all such as to favor greater percolation on the Sierra slope. Furthermore, there are

numerous springs at the base of the Sierra slopes, the flow of which, in many cases, can be measured in second-feet instead of cattle drinks. Hence, the writer still believes that the run-off coefficients used are not excessively high. Mr. Lee.

2. Charlies Butte is a low mound of lava near the margin of a shallow flow which advanced out over the valley-fill. The Butte is not the crest of a bed-rock projection, as Mr. Smith suggests, but is superimposed on the valley-fill. The spring is of the type described by the writer on page 175, and unquestionably has its source in percolation from the outwash slope.

3. The writer believes that Mr. Smith is justified in giving a value to transpiration from the luxuriant desert vegetation, the roots of which are within reach of the water-plane. The addition of this quantity would increase the computed ground-water discharge from the basin.

4. Relative to the matter of underflow from the basin past the Alabama Hills, the writer has given this matter considerable thought at various times, and cannot agree with Mr. Smith as to the probability of there being an appreciable loss at this point. For a distance of 6 miles from the "Point of the Hills" to Lone Pine Creek there are no lateral streams breaking through the hills. The whole cross-section was formerly covered by Owens Lake, and any material brought down by Hogback Creek would have been deposited on the lake shore at the "Point of the Hills", or, in case of low lake level, would have been carried directly out on the lake bed. There would be no condition favoring or even rendering possible the deposition in the section of any but the finest materials, even on the side adjoining the steep slope of the Alabama Hills. Further, there is the elevated water-plane of the Lone Pine delta opposing such underflow, and the absence of any evaporating area which would naturally result from the backwater effect. Finally, there is the fact that the valley floor of the Independence Basin above the Alabama Hills is an old lake bed beneath which fine sands and clays predominate, that the porous gravels of the outwash slope surround this relatively non-porous core, that ground-water percolating laterally through the gravels meets a barrier at the old lake shore and is compelled to seek an outlet locally by spring flow and evaporation, and that, therefore, the conditions necessary for an active underflow down stream are entirely lacking throughout the whole of the valley floor.

The discussion of probable annual variations in safe yield is not of such great importance for the Independence Basin as in other basins. This is due to two reasons, the immense storage capacity of the saturated gravels and the non-porous core or heart of the valley. The former is relatively very large with respect to any judiciously developed draft, and the latter holds up the water-plane and prevents the escape

Mr. Lee. of water from the basin by underflow. Thus, the fluctuation of the water-plane within a reasonable distance of the old lake shore would not be great, even in periods of extended drought. The effect of variations in ground-water supply in the Independence Basin would appear more as variation in spring flow and evaporation loss than as fluctuation in the water-plane.

Mr. Meinzer's long and intimate experience and scientific study of the problems involved in this paper make him one of the best qualified men in America to discuss it, and his contribution has added greatly to its value. The writer, however, feels that in general his discussion is from the point of view of the pure scientist who strives for absolute accuracy, rather than from that of the applied scientist or engineer whose aim is relative accuracy. There were involved in the solution of this particular problem not only questions of obtaining a proper internal balance in relative accuracy, but also that of giving the problem its proper place in the larger one of determining the safe yield from all sources available for a large municipal water-supply project. The quantity of ground-water available for development from the Independence Basin is about one-fifth of that available from all sources. The writer believes the results obtained, as set forth in this paper, are well within the limits of reasonable accuracy for the purpose desired. Considered in this light, he does not agree with Mr. Meinzer that the assumptions are "subject to large errors."

Considering Mr. Meinzer's discussion in detail, the writer is indebted to him for drawing attention to the erroneous inclusion with ground-water discharge of 18 sec-ft. of irrigation water dissipated by evaporation and transpiration. As has been pointed out by both Messrs. Smith and Meinzer, however, there is an appreciable loss from rank desert vegetation bordering the shallow ground-water area. The inclusion of this with ground-water losses would increase the latter, and, therefore, the corrections in the final result would tend to offset one another.

Mr. Meinzer states that observations covering 1, 2, or 3 years do not give average evaporation conditions, comparing them with precipitation and stream-gauging data in this respect. The writer does not believe that this statement would have been made if evaporation records covering several years had been examined. The range of annual variation in evaporation is usually less than 4%, whereas precipitation and stream flow may have extreme variations of more than 100 per cent.

Mr. Meinzer suggests that, in order to apply the results of the Owens Valley evaporation experiments to other valleys, it would be necessary to have a large number of observation wells and keep them under observation for several years. The writer's experience in other valleys during the past 3 years has been that water-plane fluctuations

in evaporation areas follow the same annual periodic law observed, and have about the same range as observed in Owens Valley. Hence the observation of a few judiciously chosen wells at a critical date, preferably late in September, should give dependable results. Mr. Lee.

The writer agrees heartily with Mr. Meinzer that further investigations of soil evaporation and transpiration under various conditions should be made. In connection with the rate of evaporation from bare alkaline lake beds or "playas", certain experiments recently carried on under the writer's direction tend to confirm Mr. Meinzer's observations. The evaporation from two pans of water under exactly similar conditions was observed, one pan containing distilled water and the other a sample of highly alkaline lake water. The rate of evaporation from the denser water was less than that from the fresh water, and rapidly decreased to a very small value as the salts began crystallizing out, regardless of the fact that no permanent crust was allowed to form.

The writer is very glad that Mr. Horton has emphasized the fact that in the development of hydrographic projects the most important feature, that is, water supply, is seldom given the adequate investigation that it requires. The writer has in mind several irrigation projects which are either partial or complete failures solely because insufficient study was made of the available water supply.

Although familiar with the work of the German experimenters on soil evaporation and transpiration which Mr. Horton cites, the writer found that the climatic and soil conditions under which their experiments were performed were quite different from those in Owens Valley, and that the experimental equipment did not reproduce as closely as seemed desirable the natural conditions. Hence, he found it impractical to follow their precedents as closely as would seem possible at first glance.

In comment on Mr. Horton's suggestion that tables of rainfall and run-off would have added to the value of the paper, the writer wishes to draw attention to Article VI, Section 11, of the Constitution of the Society, which contains the statement that "papers offered for presentation * * * containing matter readily found elsewhere * * * shall be rejected."

The writer takes exception to Mr. Horton's statement that there is no increment of flow during the frozen season to the mountain streams tributary to Owens Valley. Although it is true that the surface of the snow is frozen, yet on the under side, in contact with the soil, containing more or less stored heat, there is a slow but continual melting which contributes a permanent supply to stream flow all winter. This is proved by the observed fact that if freezing weather occurs before the first snowfall, the streams show material reduction in flow, but that, soon after the mountain slopes are snow-covered, there is a return to normal winter flow.

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Paper No. 1316

SUBAQUEOUS HIGHWAY TUNNELS.*

By George Duncan Snyder, M. Am. Soc. C. E.

With Discussion by Messrs. J. V. Davies and Duncan D. McBean.

Synopsis.

The object of this paper is to call attention to the use of tunnels as a means of crossing rivers or bodies of water by highways, particularly when the waters are navigable, and to discuss the conditions where tunnels are most suitable and where not.

The relative advantage of ferries, movable bridges, swinging draw-, bascule-, lift-, and transporter-bridges, are discussed, and a table showing the general dimensions of the principal transporter-bridges, is given. The suitability of pontoon bridges is considered, and also the relative advantages of fixed bridges and tunnels, and there is a table showing the general dimensions and cost of the principal bridges over navigable waters. The paper contains a history and description of existing subaqueous highway tunnels, including the Thames, Blackwall, and Rotherhithe Tunnels, in London, the Washington Street, La Salle Street, and Van Buren Street Tunnels, in Chicago; the Glasgow Harbor Tunnel, and the Hamburg Tunnel; and a table gives their general dimensions.

Projects for subaqueous highway tunnels at New York, Boston, Chicago, Sydney, Liverpool, and Oakland, Cal., are considered. Methods of construction are discussed, which include rock tunnels,

* Presented at the meeting of September 16th, 1914.

and, owing to their bearing on the subject, the Severn railway tunnel and the Mersey railway tunnel are described. Shield and compressed-air methods are described, as well as roof-shield, cut-and-cover, inside coffer-dam, and special methods, such as those used in the Harlem River crossing of the New York subway and the Detroit Tunnel. The caisson method used in Paris is considered, and also the freezing method.

Various types of lining—cast iron, brick, concrete, wrought steel, etc.—are described.

The shape of the section, means of access by inclines or shafts, methods of water-proofing, ventilation, and drainage, are also dealt with.

CONCLUSIONS.

The paper is a compilation of the available facts relative to subaqueous highway tunnels and other subaqueous tunnels applicable to highway purposes, and the conclusion is that, owing to the development of great harbors and the consequent need for unobstructed waterways and the increasing use of motor-driven vehicles, highway tunnels have not received sufficient consideration and are worthy of more extended discussion and attention.

INTRODUCTION.

With the growth of sea-borne commerce and its concentration at great ports, the problem of crossing harbors and their tributary navigable waters has become serious. The land traffic in the vicinity of a great harbor naturally develops with the water traffic, and inter-communication for land vehicles between the sides of a land-locked harbor is essential. The motor truck is being developed very rapidly and, if adequate facilities for inter-communication are provided, will develop still more rapidly and furnish a means of transferring goods from ocean vessels to railways, warehouses, and the like, thus relieving the small harbor craft and the railways from the burden of this transfer.

Where the harbor is not crowded, and with a low range of tide and a comparative freedom from fog and ice, a steam ferry is a fairly satisfactory means of crossing, as there are no excessive gradients of approach and the motors are idle or the horses resting while crossing. If, however, the harbor is crowded with water craft, there are two lines of traffic crossing each other's path on the same plane, or what in

railway parlance would be called a level or grade crossing, the avoidance of which would necessitate an under or over crossing, either by a tunnel or a bridge. With a great range of tide, it becomes difficult to load and unload boats, on account of the length of adjustable ramp necessary to avoid excessive gradients. With fog, ice, and bad weather, the danger of collision is increased, and the delays in loading, unloading, and operating ferries are serious, all of which add greatly to the cost of trucking.

On comparatively narrow channels, movable bridges of various sorts are used, but these do not avoid altogether the level crossing, as only the smaller craft can pass without having the draw-span opened. Many similar bridges are now being built at a level which will permit tugs and harbor craft to pass under them and thus decrease the frequency of opening. A clear space of 30 ft. from the surface of the water to the lowest member of a bridge will permit most harbor craft to pass under.

With a swinging draw, the large pivot pier required is a serious obstruction to the channel, and the maximum width of waterway, allowing for piers and fenders, is about 200 ft.* With a two-leaf bascule draw, the maximum span is about 300 ft. A lift-bridge has been built at Portland, Ore., with a span of 245 ft. and a clear headway of 135 ft. It will thus be seen that, at the best, such bridges interfere with the navigation of the waterway and fail to provide the free channels that an ocean port should have; also, the frequent opening of the draw obstructs the highway, and there is always the danger of a vessel colliding with a bridge.

Where the land traffic is light, transporter-bridges have been built. These are adapted to conditions where storms make the water very rough and cause difficulties in ferry navigation, or where the stream is obstructed at times by floating ice, or where a great fluctuation in the height of the water makes the design and construction of suitable landing ramps difficult. In general, they are erected at points where the importance of the river traffic is such that it cannot be hampered by a draw-bridge, where the traffic is so light as not to warrant the building of a fixed bridge or tunnel, or where the physical difficulties in the construction of a bridge or tunnel are very great.

* A swinging draw-bridge is to be constructed at Vancouver, B. C., with a length of 581¼ ft. between end bearings; this will be the longest in existence and will have a clear width of waterway of 225 ft. (*Engineering News*, February 19th, 1914.)

Table 1 gives the general dimensions of the principal transporter-bridges.

TABLE 1.—TRANSPORTER-BRIDGES.

Location.	River.	Span, in feet.	Clear height, in feet.
Widnes and Runcorn, England	Mersey	1 000	82
Newport, England	Osk	645	177
Rouen, France	Seine	644.91	167.46
Middlesbrough, England	Tees	564.75	160.9
Marseilles, France	(Harbor)	541	164.3
Bilbao, Spain	Nevrion	524.96	147.65
Nantes, France	Loire	462.62	165
Martrou, France		448.55	164.05
Rochefort, France	Charente	415	165
Duluth, Minn., U. S. A	(Harbor)	393.75	135
Kiel, Dockyards		387.2	145
Bizerte (near Tunis, Africa)		357.63	147.65
Brest, France	(Harbor)	357.6	144.4
Osten, Germany	Oste	262.5	83.8

It will be seen from Table 1 that the longest span is 1 000 ft., between Widnes and Runcorn,* over the Mersey, shown by Fig. 1. The only example in America is at Duluth, with a span of 393.75 ft.†

Most of the transporter-bridges in Table 1 are over harbors, canals, arms of the sea, or inland waters, and practically no through highway for ocean-going vessels is spanned by such a structure. There are many places where no other form of construction is warranted by the traffic; but, at the best, transporter-bridges are only suitable for comparatively narrow channels, where neither the water nor land traffic is heavy, and not for the crowded conditions of a great ocean port.

Pontoon bridges, with movable sections which can be floated to one side to permit the passing of boats, have been built. Such bridges are generally used in sparsely settled regions, where the land traffic is light; they are more or less temporary in character, and are a cheap substitute for a more permanent structure where the traffic does not warrant the heavier expenditure. However, a bridge of this type, crossing the Golden Horn, at Constantinople, which is more permanent in character, has recently been completed. The length between abutments is 1 530.84 ft., and the width of roadway, 82 ft. For small boats there are two spans of 39 ft. having a clear headway of 17½ ft., and a draw section with a clear opening of 205 ft.‡

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXV, p. 87.

† *Transactions*, Am. Soc. C. E., Vol. LV, p. 322.

‡ *The Engineer*, December 6th, 1912.

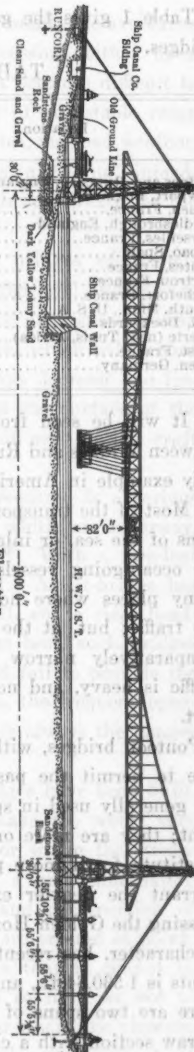
Numerous bridges of this type have been built in India.* In Calcutta one has been designed with submerged pontoons anchored at a fixed level so as to avoid the change in level of the roadway with the fluctuations of the water surface.†

Several pontoon railroad bridges of considerable length have been used in America. One, with a draw section 405 ft. long, was built at Prairie du Chien, Wis., in 1874, by the Chicago, Milwaukee and St. Paul Railway Company; this company also had four other bridges of similar type. These bridges, instead of having separate pontoons acting as piers, were really continuous floating structures.

Bridges of this type are not suited to a busy harbor frequented by numerous ocean-going vessels, and where there is a heavy land traffic.

The only remaining alternatives are fixed bridges at sufficient height to clear the shipping, or tunnels or passages at a greater depth than the draft of the vessels.

In avoiding the level crossing of these two conflicting kinds of traffic, by placing the roadway above or below the water, there is an advantage in depressing it, as the top of the structure need only be about 40 ft. below low water; whereas, if elevated, the bottom must be from 135 to 175 ft. above high water. This paper, however, is not written with the view of entering into a controversy as to the relative merits of bridges and tunnels, as no general law can



WIDNES AND RUNCORN TRANSPORTER-BRIDGE

Fig. 1. Elevation

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXIX, p. 292.

† *The Engineer*, September 13th, 1912.

be laid down; each problem must be decided on its merits in accordance with the conditions.*

Where the conditions are favorable for a bridge, there is no doubt that it is the most desirable means of crossing a body of water. These favorable conditions are high banks, which reduce the length and gradient of the approaches; moderate spans; a suitable stratum for the support of foundations at a reasonable depth; cheap land for approaches; and the ability to place piers so that they will not interfere with navigation. When the spans must be in excess of 1 000 ft., when proper support for foundations can only be had at a great depth, or when the land near the water is low, the conditions favor a tunnel. When bridges exceed a span of 1 000 ft., they must be given considerable width, for lateral stability, and the dead loads being so great, the bridges can be given great capacity by the addition of rapid transit and street railway tracks without adding greatly to the expense. This causes an undue concentration of traffic at the ends of such bridges and adds greatly to the difficulty and cost of the terminals. In other words, instead of building a bridge with capacity to suit the needs of the traffic, or several bridges located so as to suit the convenience of the public, economical considerations necessitate the construction of a single large bridge, either of greater capacity than required, or which concentrates the traffic along a single path that would be better served by several.

One other point should be given due consideration, and that is the relative ease of destruction of a bridge, as compared with a tunnel. The severing of a single member would cause the collapse of a whole span. Although injury at a single spot might cause the flooding of a tunnel, the damage would be local and could be readily repaired. This is mentioned in view of the fact that national wars have not altogether ceased, and that those who disapprove of existing conditions sometimes express their dissatisfaction by the use of explosives.

Table 2 shows the general dimensions of some of the world's principal bridges crossing waters frequented by seagoing vessels. This table does not purport to be exhaustive, but merely gives typical examples of the most important bridges of the various classes, some

* "Bridges over Navigable Rivers—Some Practical Considerations," by C. E. Smith, M. Am. Soc. C. E., Bridge Engineer, Missouri Pacific Railway, *Proceedings*, American Railway Engineering Association, Vol. XIV, Part 2, p. 185.

TABLE 2.—PRINCIPAL BRIDGES OF THE WORLD

Location.	Character.	Length of longest span, in feet.	Total length of main spans, in feet.	Total length, including approaches, in feet.	Clear height above high water, in feet.
Forth	Cantilever	1 700	5 349.5	8 298.42	150
Brooklyn, New York	Suspension	1 595.5	3 455.5	7 580	135
Manhattan, New York	"	1 470	2 920	6 855	....
Williamsburg, New York	"	1 600	2 793	7 308	135
Queensboro, New York	Cantilever	1 182	3 724.5	7 449	135
Poughkeepsie	"	548	3 093.75	6 777	130
Quebec	"	1 800	2 830	3 239	150
Tower, London	Bascule	200	880	2 680	29.5 / 143
Connecting R. R., Hell Gate	Steel arch	977.5	977.5	17 908	135
Clifton, Bristol, England	Chain suspension	702.25	....	....	245
Washington Bridge, New York	Steel arch	510	1 660	2 375	133.5
High Bridge, New York	Stone arch	80	1 450	....	100
Menai	Suspension	570	570	1 034.71	72
Britannia	Tubular	459.25	1 513	1 841.42	103.75
Saltash-Tamar R	Lenticular	436	910	2 240	100
Portland, Oregon	Lift	245	....	....	135
" "	"	220	....	....	144

being included merely on account of their historical significance. The physical conditions are such at certain of the bridges in Table 2 that a tunnel could not readily be substituted; at the Forth Bridge, on account of the great depth of water, which is more than 200 ft.; at the connecting railroad bridge at Hell Gate, for the same reason; at the Poughkeepsie Bridge, Washington Bridge, High Bridge, etc., on account of the height of the river banks. On the other hand, at many of the other bridge locations, in view of the modern state of the art, tunnels could be constructed more advantageously than bridges.*

HIGHWAY TUNNELS: HISTORY.

London.—It is a curious fact that one of the first subaqueous tunnels, and the first on which a shield was used, was constructed as a highway tunnel, but was never used for that purpose. This was the first Thames Tunnel, built by the inventor of the shield, the late Sir Marc Isambard Brunel. His patent for a shield was No. 4 204, of 1818. Owing to London Bridge marking the head of navigation

* "Bridges and Ferry Bridges; Tunnels under Waterways used for Ocean Navigation—Economic and Technical Study," a report by Baurat Wendemuth, International Congress of Navigation, 1912.

OVER WATERS FREQUENTED BY SEAGOING VESSELS.

Width, in feet.	Use.	Dates.		Depth of water, in feet.	Cost.	Gradient of approaches.
		Work begun.	Work finished.			
30	Railway.	1883	1890	200	$15 700 000	1.43%
86	Highway, foot-way, electric railway, trolley.	Jan. 3, 1870	May 24, 1883	62	22 400 000	3.25%
120	"	Oct. 1, 1901	Dec. 31, 1909	67	26 000 000	3.25%
118	"	Nov. 7, 1896	Dec. 19, 1903	50	23 100 000	3.00%
89.5	"	June, 1901	Mar. 30, 1909	88	17 900 000	3.50%
35	Railway.	Sept., 1886	1889	60	....	....
88	"	....	....	200	(Disaster, Aug. 29th, 1907.)	
50	Highway.	June, 1886	June, 1894	33.5	....	....
93	Railway.	July, 1912	....	108	....	1.20%
31	Highway.	1861	Dec., 1864	31	$360 000	....
80	"	July, 1886	Feb., 1889	14.5	2 851 684	....
21	Aqueduct.	....	1842	14.5	....	....
29.5	Highway.	1819	1826	30	....	Level.
2-14.5	Railway.	Apr. 13, 1846	1850	75	2 931 000	"
16.83	"	....	May, 1859	60	....	....
....	Highway.	....	1911	....	....	....
....	Highway and railway.	....	....	....	....	....

for seagoing vessels, the development of the docks and tributary industries was necessarily below this point, so that the need of some means of crossing the river below London Bridge is evident. This is indicated in Fig. 2, which shows the position of the Tower Bridge and the various tunnels below London Bridge. The Thames Tunnel, Fig. 3, crosses the river between Rotherhithe and Wapping, about 1½ miles below London Bridge. Work was started in 1825 and, after being suspended several times owing to physical and financial difficulties, was completed in 1843. In section it is one of the widest openings ever constructed, the total width of excavation being 37 ft. 6 in., and in height 22 ft. 3 in., separated by a wall 4 ft. thick, pierced by 64 arched openings, each of 4 ft. span. The length is 1 200 ft. It was proposed to have spiral approaches, but this was never done; the construction shafts, however, were fitted with stairways. In 1866, the tunnel was sold to the East London Railway Company, and since that time this company has operated its trains through it. Its cost, including shafts, was £1 300 per lin. yd.

As the Thames Tunnel had been diverted from its intended use, the vehicular traffic still had no facilities below London Bridge, but its needs were partly met by the Tower Bridge, about ½ mile below

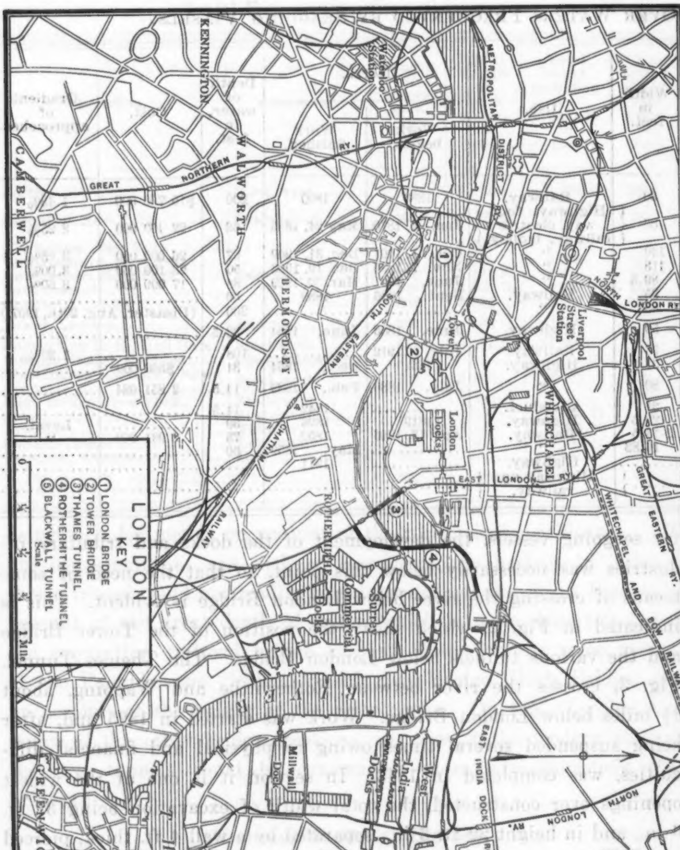
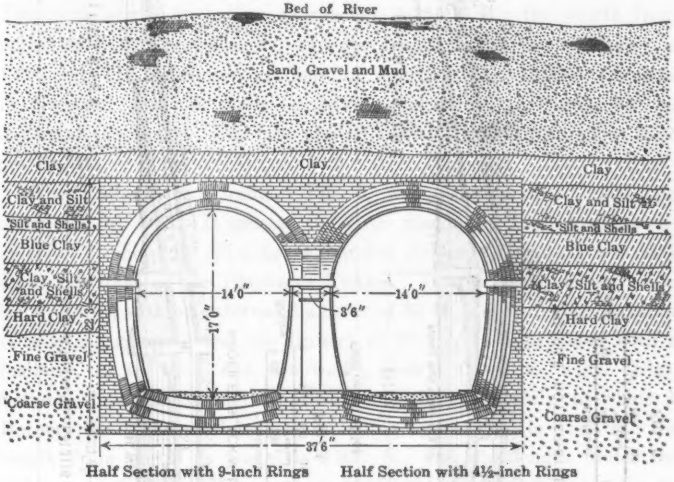
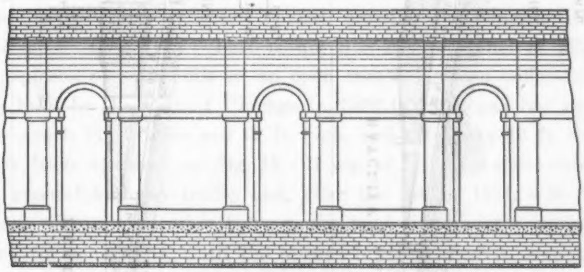

LONDON
KEY
1 LONDON BRIDGE
2 TOWER BRIDGE
3 THAMES TUNNEL
4 ROTHERHITHE TUNNEL
5 BLACKWALL TUNNEL
Scale
0
1 Mile
KENNINGTON
WALWORTH
CAMBERWELL
BERMONDSEY
ROTHERHITHE
GREENWICH
WHITECHAPEL
Waterloo Station
Liverpool Street Station
Tower
London Docks
Commercial Docks
Surrey
West India Docks
Millwall Dock
East India Docks
Victoria Dock
Greenwich Park
Greenwich Marshes
West Ham Abbey Marshes
METROPOLITAN DISTRICT RY.
NORTH LONDON RY.
EAST LONDON RY.
GREAT EASTERN RY.
WHITECHAPEL AND BOW RAILWAY
GREAT NORTHERN RY.
SOUTH EASTERN
CHATHAM RAILWAY
EAST INDIA DOCK ROAD

FIG. 2.

London Bridge, opened to traffic in 1894. This consists of a double-leaf bascule draw, with a clear span between towers of 200 ft., and a high-level footbridge connecting the towers, leaving a clear head-room of 139½ ft. above high water.



CROSS-SECTION

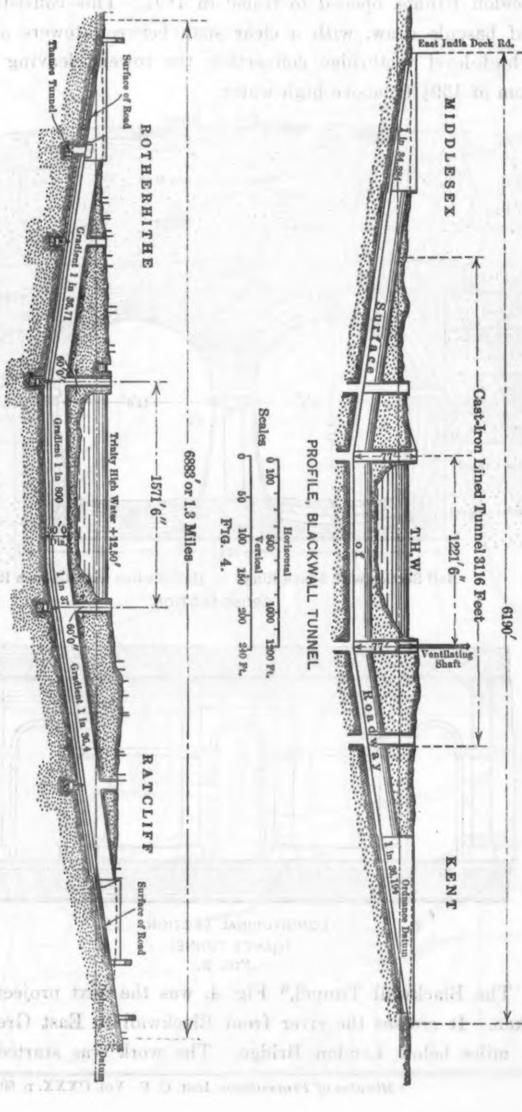

LONGITUDINAL SECTION

THAMES TUNNEL

FIG. 3.

The Blackwall Tunnel,* Fig. 4, was the next project to be undertaken. It crosses the river from Blackwall to East Greenwich, about 6¼ miles below London Bridge. The work was started in 1891 and

* *Minutes of Proceedings*, Inst. C. E., Vol. CXXX, p. 50.



PROFILE, BLACKWALL TUNNEL

FIG. 4.

PROFILE, ROTHERHITHE TUNNEL

FIG. 5.

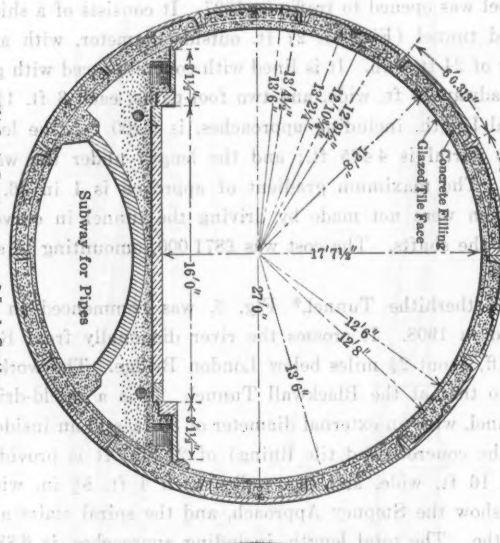
the tunnel was opened to traffic in 1897. It consists of a shield-driven, iron-lined tunnel (Fig. 6), 27 ft. outside diameter, with an internal diameter of 24 ft. 3 in. It is lined with concrete faced with glazed tile, has a roadway 16 ft. wide, and two foot-paths, each 3 ft. 1½ in. wide. The total length, including approaches, is 6 200 ft.; the length from portal to portal is 4 465 ft.; and the length under the waterway is 1 220 ft. The maximum gradient of approach is 1 in 36. Changes in direction were not made by driving the tunnel in curves, but by angles at the shafts. The cost was £871 000, amounting to about $685 per lin. ft.

The Rotherhithe Tunnel,* Fig. 5, was commenced in 1904 and completed in 1908. It crosses the river diagonally from Rotherhithe to Ratcliff, about 2¼ miles below London Bridge. The work was very similar to that at the Blackwall Tunnel. It is a shield-driven, iron-lined tunnel, with an external diameter of 30 ft. and an inside diameter (inside the concrete and tile lining) of 27 ft. It is provided with a roadway, 16 ft. wide, and two walks, each 4 ft. 8½ in. wide. Figs. 8 and 9 show the Stepney Approach, and the spiral stairs at Shaft 2, Rotherhithe. The total length, including approaches, is 6 883 ft.; the length from portal to portal is 4 930 ft.; and the length under the waterway is 1 480 ft. The cost was £1 088 484, or about $930 per lin. ft.

Chicago.—In the United States the first subaqueous highway tunnel was the Washington Street Tunnel, crossing the Chicago River, in Chicago.† (Fig. 10.) This is not properly a tunnel at all, but a subaqueous passage built in an open trench between coffer-dams. It was built by the City of Chicago in 1866 to 1869, and had two roadways, each 11 ft. wide and 13 ft. high, and a footway 10 ft. wide and 10 ft. high, as shown by Fig. 11. It was at first used quite extensively for general highway traffic, and, after the fire in 1871, was the only means of communication between the west side and the business district, for some time, until the burned bridges could be restored. Incidentally, a tunnel is less likely to be damaged by fire or flood than is a bridge. This tunnel, later, fell into a bad state of repair, and was used much less frequently, the traffic being largely diverted to the nearby bridges. At the time of the change of power from horse

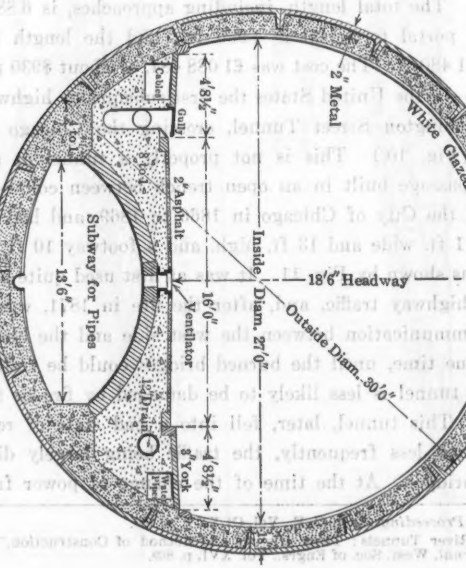
* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXV, p. 190.

† "Chicago River Tunnels: Their History and Method of Construction," by William Artingstall, *Journal*, West. Soc. of Engrs., Vol. XVI, p. 869.



CROSS-SECTION, BLACKWALL TUNNEL

FIG. 6.



CROSS-SECTION, ROTHERHITHE TUNNEL

FIG. 7.

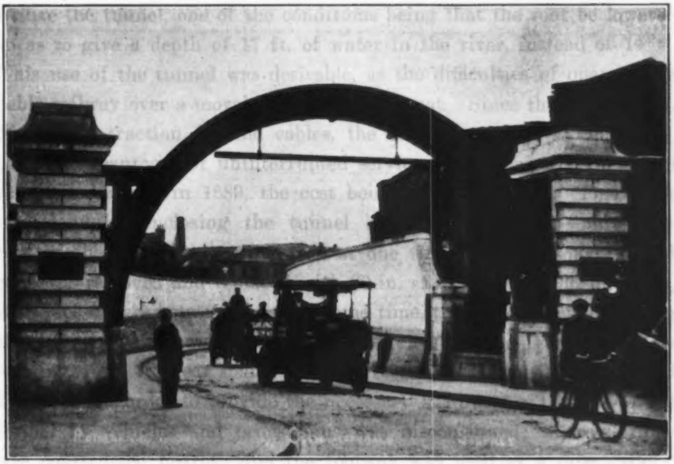
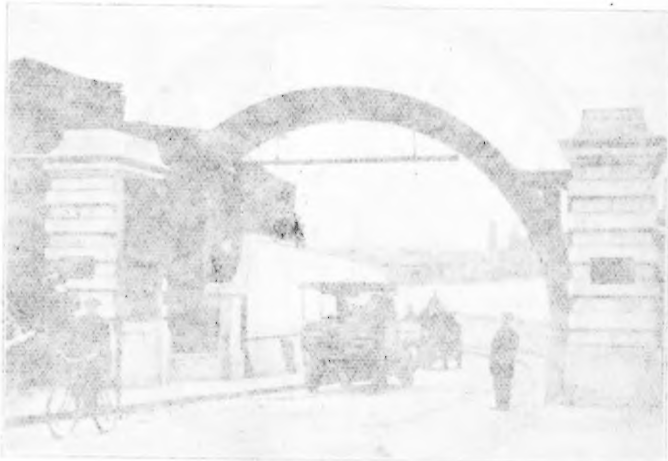
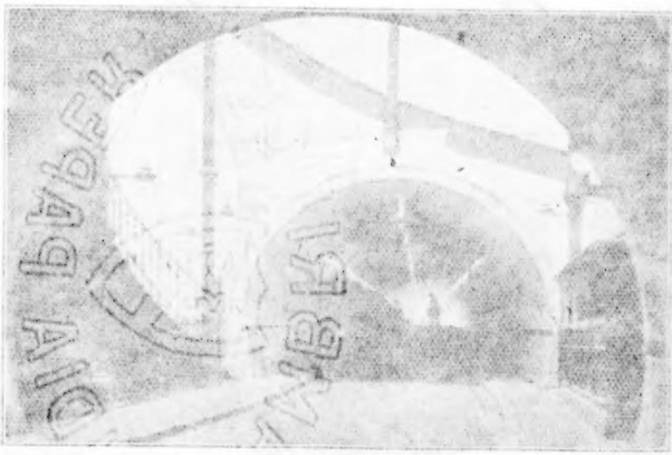


Fig. 8.—Rotherhithe Tunnel: Open Approach, Stepney.

Fig. 9.—Rotherhithe Tunnel: Spiral Stairs, Shaft 2.



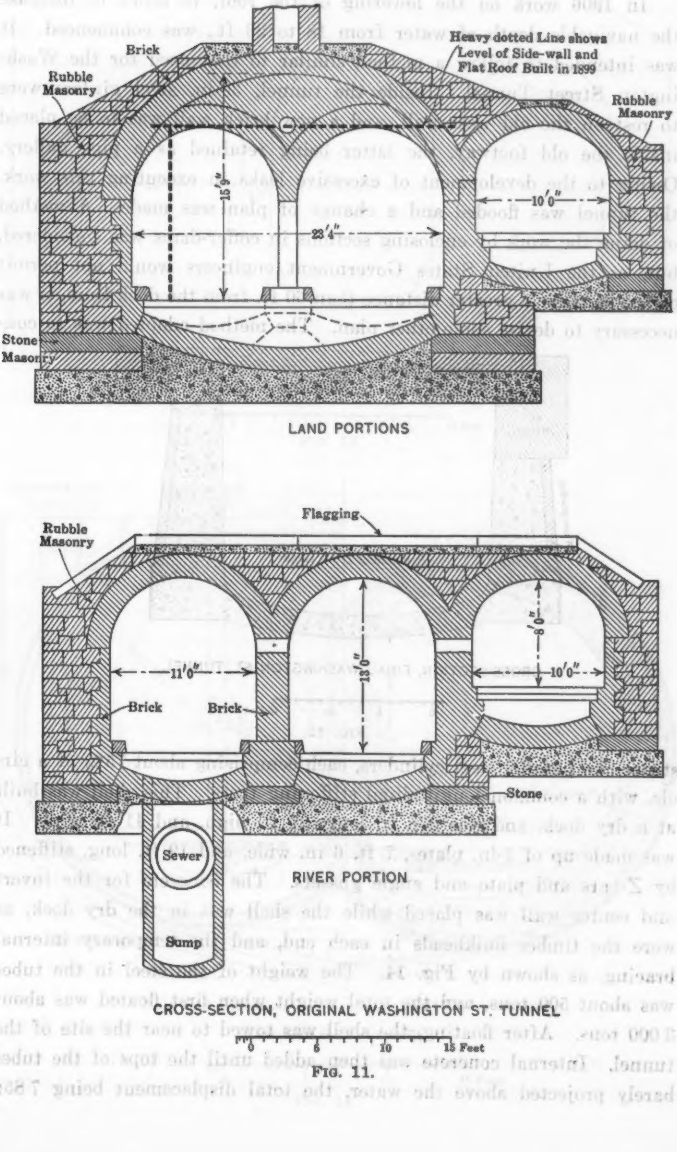


to cable on the Chicago street railways, permission was obtained to utilize the tunnel, one of the conditions being that the roof be lowered so as to give a depth of 17 ft. of water in the river, instead of 14 ft. This use of the tunnel was desirable, as the difficulties of operating a cable railway over a movable bridge were great. Since the substitution of electric traction for the cables, the tunnels are not so necessary, but the advantages of uninterrupted service still remain. The tunnel roof was lowered in 1889, the cost being about $135 000. This work was done by enclosing the tunnel between coffer-dams—only one-half of the river being obstructed at one time. The masonry arched roof was removed and replaced with 20-in. steel I-beams, 36 in. apart, with jack-arches between. At the same time, the dividing wall between the two roadways was removed and the tunnel was given a single opening of 20 ft. clear span. The invert was also cut out and lowered 2 ft. 6 in. in order to provide the necessary headroom. The use of the tunnel by vehicles and pedestrians was discontinued; a draw-bridge was constructed nearby; and the footway was used as a pipe gallery. In 1906 the tunnel was again reconstructed, in order to provide a navigable depth of 26 ft. instead of 17 ft. This was accomplished by cutting chases in the walls and inserting 33-in. plate girders, 48 in. apart, supporting a new masonry roof, after which the old roof was removed by breaking it up, without blasting, and dredging from the exterior. The side-walls were then underpinned and a new invert was constructed, as shown by Fig. 12. In 1909 further alterations were made in order to enable the tunnel eventually to form part of a subway system in the business district, and this necessitated building temporary inclined approaches to enable the surface cars to continue the use of the tunnel, pending the construction of the subway system. The cost of this work was about $675 000.

La Salle Street Tunnel (*Chicago*).—The construction of the original La Salle Street Tunnel was commenced by the City of Chicago in 1869, and it was opened for traffic on July 4th, 1871. The type of construction was similar to that of the Washington Street Tunnel. The total length is 1 887 ft., and the gradients of approach are 1 in 20. In section, it consisted of two roadways, each 11 ft. wide, and a footway 11 ft. wide and 10 ft. high, as shown by Fig. 13. The original cost was $566 000. The tunnel was given over to cable-operated street cars in 1889.

CHICAGO
KEY
① LA SALLE ST. TUNNEL
② WASHINGTON ST. TUNNEL
③ VAN BUREN ST. TUNNEL
Scale
400 0 400 800 1200 1600 Feet
N. HALSTED ST.
N. CLARK ST.
LA SALLE ST.
NORTH BRANCH
INDIANA ST.
MICHIGAN ST.
KINZIE ST.
RIVER
CHICAGO
CLINTON ST.
N. CANAL ST.
LAKE ST.
RANDOLPH ST.
WASHINGTON ST.
MADISON ST.
MARKET ST.
FRANKLIN ST.
W. JACKSON AVE.
W. VAN BUREN ST.
VAN BUREN ST.
W. HARRISON AVE.
FIFTH AVE.
LA SALLE ST.
DEARBORN ST.
SOUTH BRANCH
GRANT PARK
LAKE MICHIGAN
S. HALSTED ST.
CANAL ST.
S. CLARK ST.
STATE ST.
MICHIGAN AVE.

FIG. 10.



CROSS-SECTION, ORIGINAL WASHINGTON ST. TUNNEL

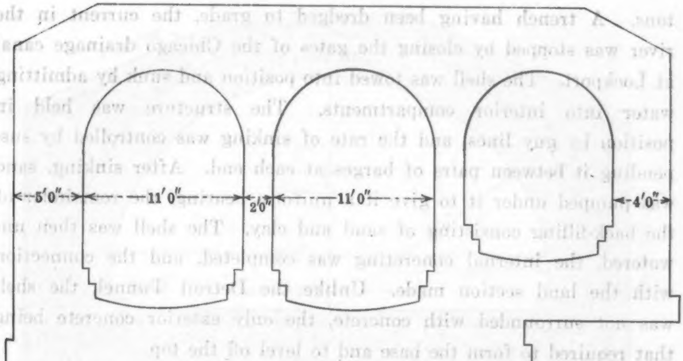
FIG. 11.

In 1906 work on the lowering of the roof, in order to increase the navigable depth of water from 18 to 26 ft., was commenced. It was intended to follow a method similar to that used for the Washington Street Tunnel. Inside the tunnel, 30-in. steel girders were to rest on the old west wall, and a new brick wall was to be placed inside the old footway, the latter being retained as a pipe gallery. Owing to the development of excessive leaks in executing this work, the tunnel was flooded and a change of plan was made. A method of doing the work by enclosing sections in coffer-dams was considered, but, as the United States Government engineers would not permit coffer-dams for a greater distance than 60 ft. from the dock lines, it was necessary to devise some other plan. The method adopted was to con-

CROSS-SECTION, FINAL WASHINGTON ST. TUNNEL

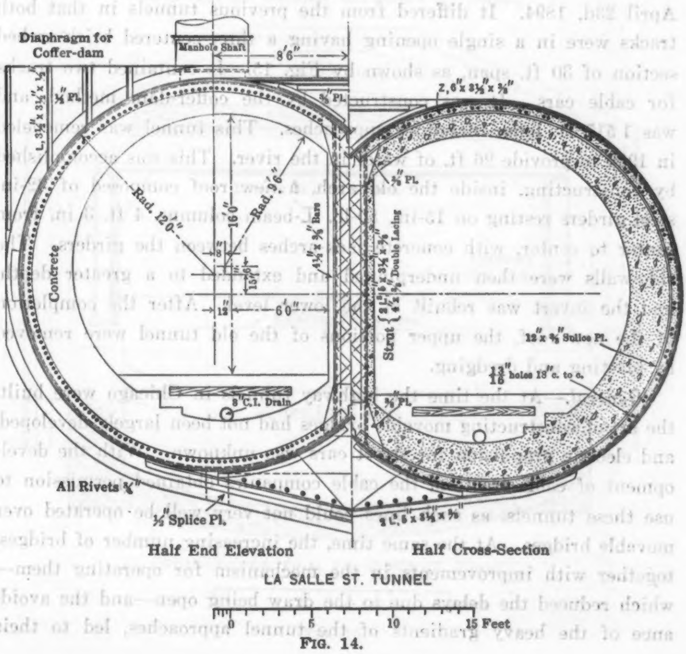
FIG. 12.

struct two parallel steel cylinders, each comprising about 75% of a circle, with a common longitudinal stiffening truss. This shell was built at a dry dock, and was 278 ft. long, 27 ft. high, and 41 ft. wide. It was made up of ⅜-in. plates, 7 ft. 6 in. wide, and 19 ft. long, stiffened by Z-bars and plate and angle gussets. The concrete for the invert and center wall was placed while the shell was in the dry dock, as were the timber bulkheads in each end, and the temporary internal bracing, as shown by Fig. 14. The weight of the steel in the tubes was about 500 tons, and the total weight when first floated was about 3 000 tons. After floating, the shell was towed to near the site of the tunnel. Internal concrete was then added until the tops of the tubes barely projected above the water, the total displacement being 7 852



CROSS-SECTION, ORIGINAL LA SALLE ST. TUNNEL

FIG. 13.



LA SALLE ST. TUNNEL

FIG. 14.

tons. A trench having been dredged to grade, the current in the river was stopped by closing the gates of the Chicago drainage canal at Lockport. The shell was towed into position and sunk by admitting water into interior compartments. The structure was held in position by guy lines, and the rate of sinking was controlled by suspending it between pairs of barges at each end. After sinking, sand was pumped under it to give it a uniform bearing, the remainder of the back-filling consisting of sand and clay. The shell was then unwatered, the internal concreting was completed, and the connection with the land section made. Unlike the Detroit Tunnel, the shell was not surrounded with concrete, the only exterior concrete being that required to form the base and to level off the top.

Van Buren Street Tunnel (Chicago).—The Van Buren Street Tunnel was built by the West Chicago Street Railway Company. Work was started in 1889, and the tunnel was opened to traffic on April 23d, 1894. It differed from the previous tunnels in that both tracks were in a single opening having a three-centered brick-arched section of 30 ft. span, as shown by Fig. 15. It contained two tracks for cable cars. It was constructed by the coffer-dam method, and was 1 517 ft. long, including approaches. This tunnel was remodeled in 1906 to provide 26 ft. of water in the river. This was accomplished by constructing, inside the old arch, a new roof composed of 32-in. steel girders resting on 15-in., 80-lb. I-beam columns, 4 ft. 3 in. from center to center, with concrete jack-arches between the girders. The side-walls were then underpinned and extended to a greater depth, and the invert was rebuilt at the lower level. After the completion of the new roof, the upper portions of the old tunnel were removed by blasting and dredging.

General.—At the time the highway tunnels in Chicago were built, the art of constructing movable bridges had not been largely developed, and electric propulsion for street cars was unknown. With the development of cable traction, the cable companies obtained permission to use these tunnels, as such lines could not very well be operated over movable bridges. At the same time, the increasing number of bridges, together with improvements in the mechanism for operating them—which reduced the delays due to the draw being open—and the avoidance of the heavy gradients of the tunnel approaches, led to their

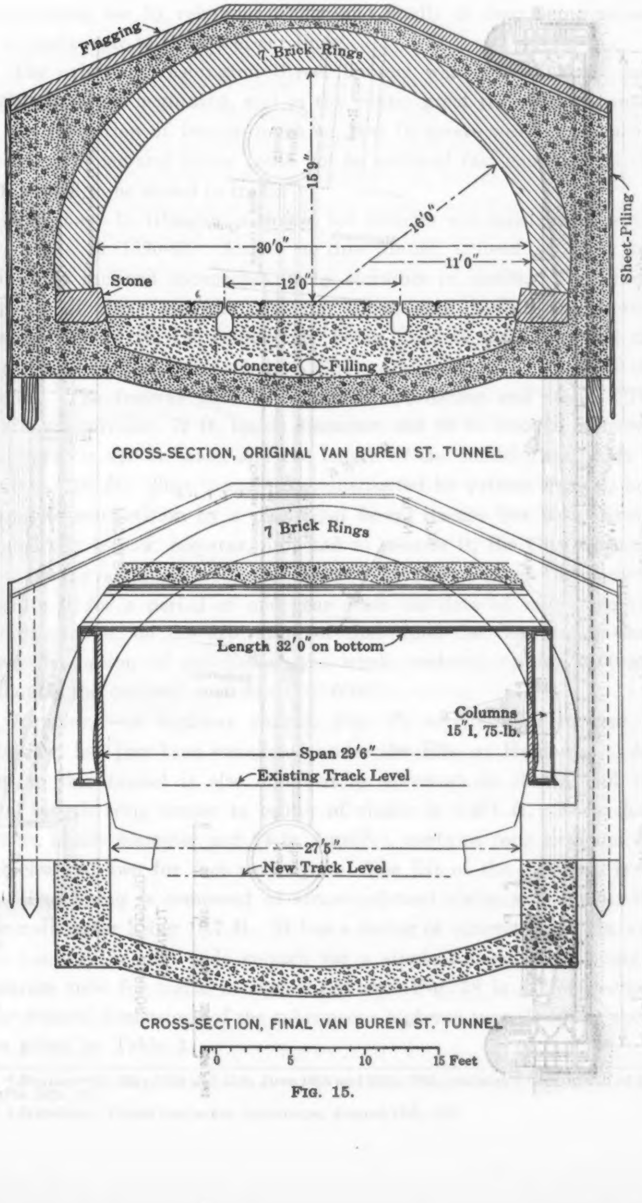
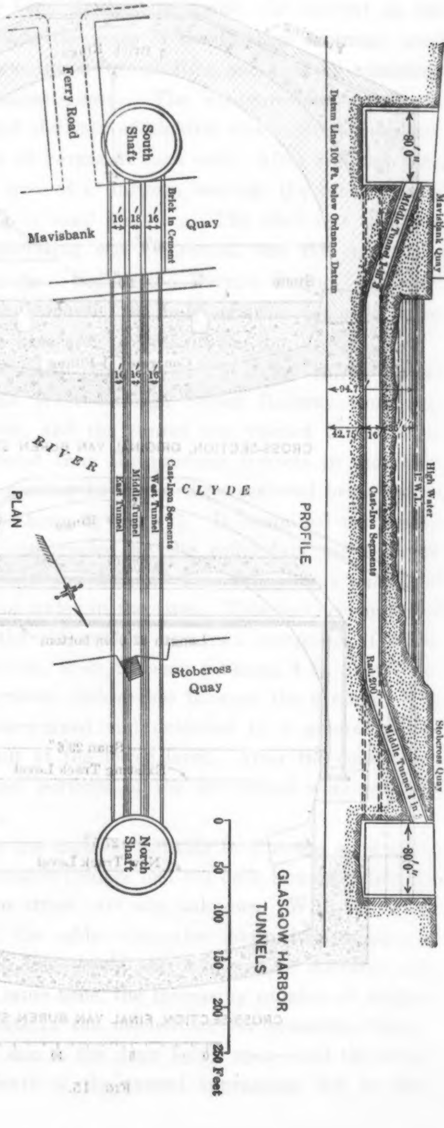

Flagging
7 Brick Rings
15'9"
16'0"
30'0"
11'0"
Stone
12'0"
Sheet-Piling
Concrete Filling
CROSS-SECTION, ORIGINAL VAN BUREN ST. TUNNEL
7 Brick Rings
Length 32'0" on bottom
Columns
15" I, 75-lb.
Span 29'6"
Existing Track Level
27'5"
New Track Level
CROSS-SECTION, FINAL VAN BUREN ST. TUNNEL
0
5
10
15 Feet

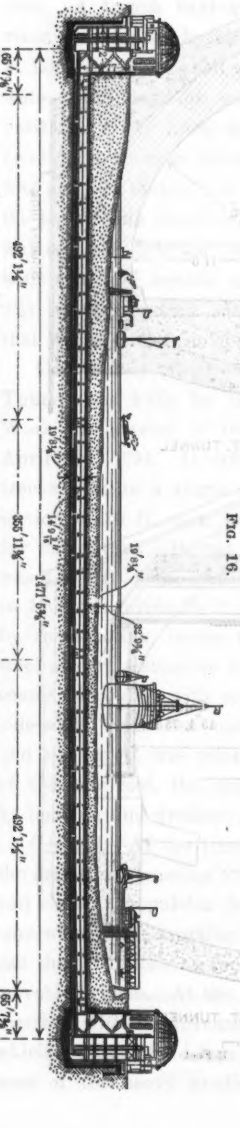
FIG. 15.



GLASGOW HARBOR TUNNELS

PLAN

FIG. 16.



PROFILE OF ELBE TUNNEL AT HAMBURG

FIG. 17.

diminishing use by vehicular traffic, and finally to their being turned over exclusively to the street-car lines.

The original Washington Street Tunnel was in very leaky condition from the beginning, and in the winter great trouble was caused by the formation of ice—so much so, that in severe weather the accumulation of ice and icicles could not be removed fast enough, and the tunnel had to be closed to traffic.

Glasgow.—In Glasgow, a tunnel for vehicles was constructed across the Clyde in 1890-93. Access to this tunnel, instead of being obtained by inclined approaches, is by elevators in shafts. The tunnel consists of three separate cast-iron tubes, each having an internal diameter of 16 ft., the center tube being used as a footway and the outer ones for vehicles, each in a single direction. Fig. 16 shows the profile. The footway tube has inclined approaches and stairs. The shafts are circular, 76 ft. inside diameter, and 80 ft. outside diameter. The river is 415 ft. wide, and the length of the tunnel from shaft to shaft is 700 ft. This tunnel was constructed by private capital, but, owing to competition by a municipal ferry, its use has been discontinued. It is now, however, proposed to re-open it, the City Corporation having arranged with the Glasgow Harbor Tunnel Company to operate it for a period of one year after the date of re-opening, on condition that, on the expiration of that time, the Corporation shall have the option of purchasing the whole undertaking for £100 000, although the original cost was £287 000.*

Hamburg.—A highway tunnel, Fig. 17, very similar to that at Glasgow, has just been completed under the Elbe at Hamburg.† Access to this tunnel is also obtained by elevators in shafts, and the total length from center to center of shafts is 1 471 ft. Each shaft (72 ft. inside diameter and 84 ft. outside), contains four elevators for vehicles and two for foot passengers. The lift of the elevators is 78 ft. The lining is composed of structural-steel plates and angles, the outer diameter being 19.7 ft. It has a lining of concrete and tile, and the roadway is only wide enough for a single vehicle, there being a separate tube for traffic in each direction. Fig. 18 is a cross-section. The general dimensions of the subaqueous highway tunnels of the world are given in Table 3.

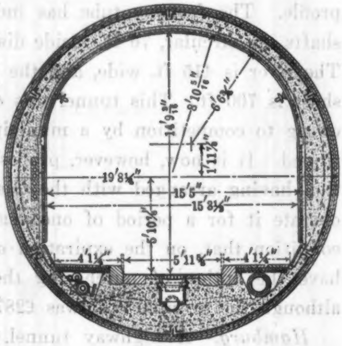
* *Engineering*, May 10th and 31st, June 14th and 28th, 1895, contains a description of the shafts, lifts, etc.

† *Zeitschrift*, Verein Deutscher Ingenieure, August 17th, 1912.

Projects.

From the preceding it will be seen that highway tunnels have not had a very fortunate history. The Thames Tunnel, the first ever built, was never utilized for the purpose intended, but was first used by a steam railway, and now by a railway operated by electric power. The tunnel, however, is now performing a useful function, and is practically in as good condition as when completed, 70 years ago, carrying more vehicles, each of much greater weight, than was contemplated originally, and it is not probable that as much could be said of a bridge, other than one of masonry.

The Chicago tunnels have had to be rebuilt to provide greater depth of water (one of them being rebuilt twice), and their use for general highway traffic has been discontinued. They are now being utilized only by the street cars.



CROSS-SECTION, OF ELBE TUNNEL AT HAMBURG

Fig. 18.

The use of the Glasgow Tunnel has been discontinued, owing to municipal competition, and only the Blackwall, Rotherhithe, and Hamburg Tunnels remain in successful operation. Nevertheless, various projects are under consideration for the construction of highway tunnels, and therefore the subject is worthy of consideration.

New York.—Some means other than ferries for vehicles to cross the Hudson (North) River at New York has long been needed. As the river forms the boundary between the States of New York and New Jersey, such means of crossing must be provided by the joint action of the two States or by a private corporation. This subject is now under investigation by the two States, through the medium of the New Jersey Interstate Bridge and Tunnel Commission and the New York Interstate Bridge and Tunnel Commission. These two Commissions have agreed that, if a bridge is to be built, it should be as far south as physical conditions will permit. As no suitable foundation

can be obtained at points where piers would be permitted in the lower reaches of the river, a location at about Fifty-eighth Street has been recommended. This will require a span of 2 880 ft.

These Commissions have recommended a location for a tunnel crossing the river from the foot of Canal Street, New York, to about the foot of Thirteenth Street, Jersey City, as shown on the map, Fig. 19. This location is the most suitable, as it is in the average path of the present lines of communication for vehicles crossing the river, and is within easy reach of the present bridges over the East River. The New York approach to such a tunnel will come to the surface at about the intersection of Varick and Canal Streets. Varick Street is now being widened, and will connect with the extension of Seventh Avenue, which is now under way.

The present traffic crossing the river at New York is about 20 000 vehicles daily.

Boston.—In Boston there has been some agitation for a highway tunnel from Boston to East Boston. Gen. J. G. Foster, when United States Engineer Officer at that port, in 1868, prepared a design for such a tunnel.

Chicago.—In Chicago it has been proposed to build a tunnel under the Chicago River in order to connect the Boulevard and Park System. Two tunnels, each 31 ft. in diameter, have been suggested.

Sydney.—At Sydney, New South Wales, a highway tunnel has been proposed to connect Sydney and North Sydney. The subaqueous portion would be 1 416 ft. long, and the total length about 1⅓ miles. The proposed section would be about 21 by 27½ ft. inside, with a 16-ft. roadway and two 4½-ft. walkways. The more recent recommendations, however, have been for a bridge.

Liverpool.—It has recently been proposed to connect Liverpool and Birkenhead by a tunnel under the Mersey for the use of vehicles and tram cars. As a successful passenger railway tunnel has been constructed under the river at this point, there would probably be no insurmountable physical difficulty in building a similar tunnel for highway purposes.

Oakland, Cal.—A tunnel has been proposed between Oakland and Alameda for a 20-ft. driveway and two 6-ft. suspended foot-walks.

TABLE 3.—GENERAL DIMENSIONS OF THE

Location.	Method of construction.	Dates.		Length under water-way, in feet.	Length, portal to portal, in feet.	Total length including approaches, in feet.	Depth to invert, in feet.
		Work begun.	Work finished.				
Thames	Shield	1825	1843	925	1 200	......	65
Blackwall	Shield	1891	1897	1 220	4 465	6 200	80
Rotherhithe	Shield	1904	1908	1 460	4 930	6 883	75
Chicago, Washington St.	Coffer-dam	1866	1869	152	934	1 608	36
Chicago, Washington St.	Reconstructed	1906	1910	152	......	1 550	53
Chicago, La Salle St.	Coffer-dam	1869	1871	285	1 505	1 890	38
" " "	Reconstructed	1906	....	285	.....	2 030	50
" Van Buren St.	Coffer-dam	1889	1894	184	920	1 514	44
Glasgow	Shield	1890	1893	415	700	......	65
Hamburg	Shield	1907	1911	1 214	1 471	......	72
Severn	Rock	1873	July 1, 1887	2¼ miles, 1 650 ft.	22 998	6½ miles	156

METHODS OF CONSTRUCTION.

Existing highway tunnels have been constructed in soft ground by the shield method, by coffer-dam, or by sinking a metal form in a trench dredged from the surface, in which a lining of concrete was deposited; but, as other methods have been used successfully in building subaqueous tunnels for other purposes, they can be applied to the construction of highway tunnels where the conditions are favorable.

All the European examples are shield-driven. The first Thames Tunnel was driven without compressed air, but it was used on the Blackwall, Rotherhithe, Glasgow Harbor, and Hamburg Tunnels. In the United States, the Chicago tunnels were originally built within coffer-dams and afterward lowered by various means; in the La Salle Street Tunnel, a metal form was sunk from the surface.

Rock.—No subaqueous highway tunnels have been driven in rock formation, but several notable tunnels for other purposes have been constructed in rock, and there is no reason that they could not be built for highway purposes where the conditions are favorable. Both the Mersey and Severn Tunnels were driven in rock without the aid of compressed air. The Mersey Tunnel has an internal span of 26 ft. and a height of 23 ft. from the invert to the intrados. It is lined with brick masonry, and contains a double-track railway. It was opened on

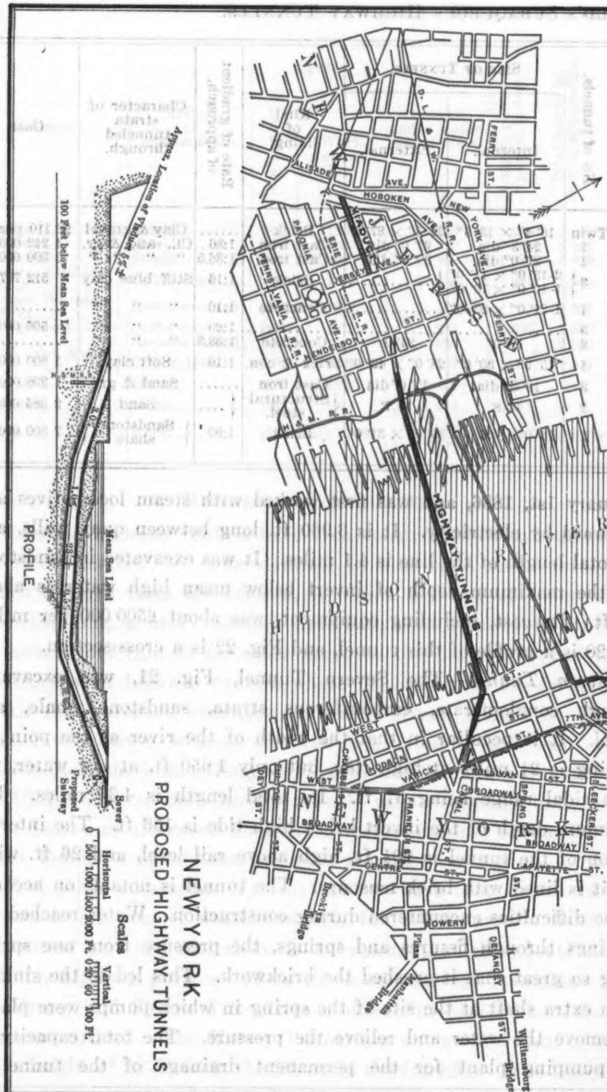
WORLD'S SUBAQUEOUS HIGHWAY TUNNELS.

Depth of water, in feet.	No. of tunnels.	Size of Tunnels. Internal.	Size of Tunnels. External.	Kind of lining.	Rate of gradient of approach.	Character of strata tunneled through.	Cost.
32	Twin	16′ 4″ × 13′ 9″	22′ 3″ × 37′ 6″	Brick	......	Clay & gravel	$2 110 per ft.
40	1	24′ 3″ dia.	27′ 0″ dia.	Cast iron	1:36	Cl., sand & gr.	4 242 000
40	1	27′ 0″ dia.	30′ 0″ dia.	Cast iron	1:36.5	" " " "	5 300 000
14	3	2–13′ 0″ × 11′ 0″ 1–10′ 0″ × 10′ 0″	....	Brick	1:16	Stiff blue clay	512 707
26	1	1–21′ 0″ × 25′ 0″	....	Concrete	1:10	" " "	....
17	3	....	....	....	1:20	" " "	566 000
27	2	19′ 0″	24′ 0″	Concrete	1:33.3	" " "	....
18	1	15′ 9″ × 30′ 0″	28′ 0″ × 42′ 0″	Brick & con.	1:10	Soft clay	1 800 000
49	3	16′ 0″ dia.	17′ 0″ dia.	Cast iron	......	Sand & gr.	1 398 000
33	2	14.8′	19.7′	Structural steel.	....	Sand	2 386 000
96.30	1	25′ 0″ × 26′ 0″	30′ 0″ × 31′ 0″	Brick	1:90	Sandstone shale	7 800 000

February 1st, 1886, and was first worked with steam locomotives and afterward by electricity. It is 3 960 ft. long between quay walls, and the total length of the line is 4.1 miles. It was excavated in sandstone, and the maximum depth of invert below mean high water is about 151 ft. Its cost, including equipment, was about £500 000 per mile.* Fig. 20 is a profile of this tunnel, and Fig. 22 is a cross-section.

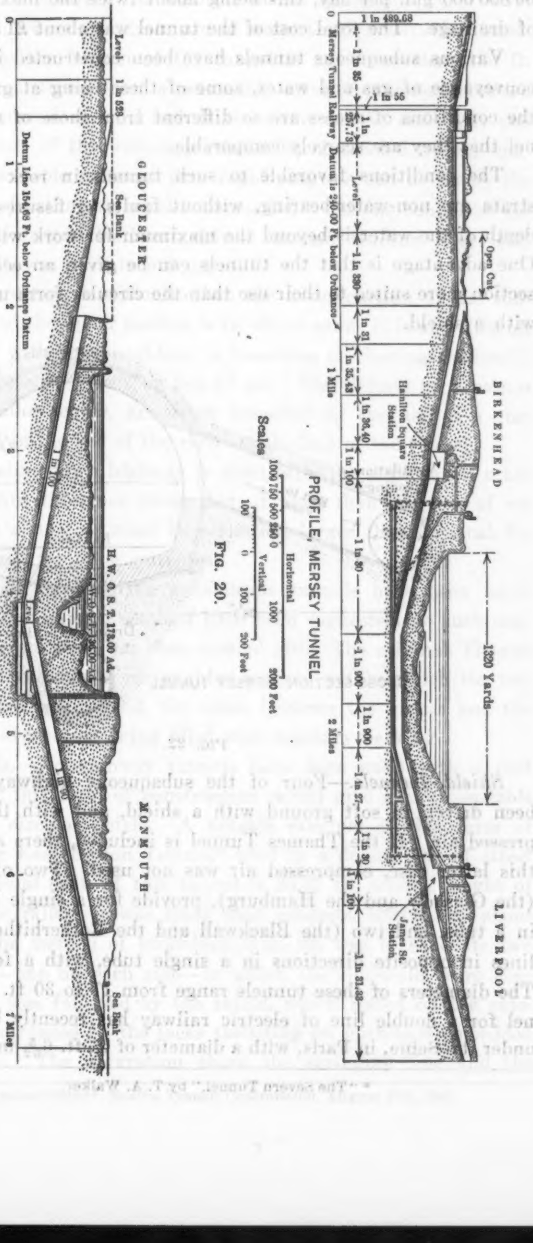
Severn Tunnel.—The Severn Tunnel, Fig. 21, was excavated through conglomerate, carboniferous strata, sandstone, shale, and gravel. It is peculiar in that the width of the river at the point of crossing is 2¼ miles at high tide, but only 1 650 ft. at low water, the mean tidal range being 37 ft. Its total length is 4.36 miles. The maximum depth of the invert below high tide is 156 ft. The internal section of the tunnel is 20½ ft. high above rail level, and 26 ft. wide, and it is lined with brick masonry. The tunnel is notable on account of the difficulties encountered during construction. Water reached the workings through fissures and springs, the pressure from one spring being so great that it crushed the brickwork. This led to the sinking of an extra shaft at the site of the spring in which pumps were placed to remove the water and relieve the pressure. The total capacity of the pumping plant for the permanent drainage of the tunnel is

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXXVI, p. 40.



NEW YORK
PROPOSED HIGHWAY TUNNELS

FIG. 19.



PROFILE, MERSEY TUNNEL

FIG. 20.

PROFILE, SEVERN TUNNEL

FIG. 21.

66 000 000 gal. per day, this being about twice the maximum quantity of drainage. The total cost of the tunnel was about £1 500 000.*

Various subaqueous tunnels have been constructed in rock for the conveyance of gas and water, some of these being at great depth, but the conditions of access are so different from those of a highway tunnel that they are scarcely comparable.

The conditions favorable to such tunnels in rock are where the strata are non-water-bearing, without faults or fissures, or where the depth of the water is beyond the maximum for work with air pressure. One advantage is that the tunnels can be given an economical cross-section more suited to their use than the circular form usually obtained with a shield.

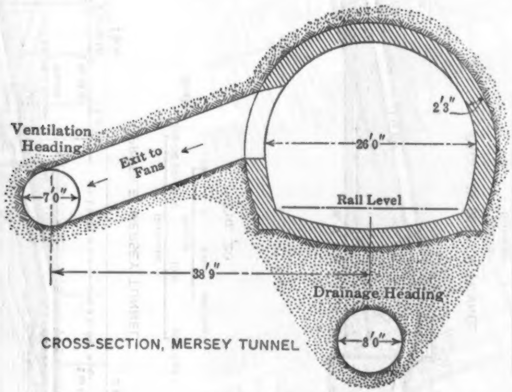


FIG. 22.

Shield Tunnels.—Four of the subaqueous highway tunnels have been driven in soft ground with a shield, and with the aid of compressed air. If the Thames Tunnel is included, there are five, but, in this latter case, compressed air was not used. Two of these tunnels (the Glasgow and the Hamburg), provide for a single line of vehicles in a tube, and two (the Blackwall and the Rotherhithe), provide two lines in opposite directions in a single tube, with a footway as well. The diameters of these tunnels range from 17 to 30 ft. A shield tunnel for a double line of electric railway has recently been completed under the Seine, in Paris, with a diameter of 25 ft. $6\frac{5}{32}$ in.

* "The Severn Tunnel," by T. A. Walker.

The conditions favoring the selection of the shield method of driving tunnels are: where the strata are of water-bearing silt, clay, sand, or gravel; where the bottom of the tunnel is not more than 100 ft. below the water surface; where a reasonable cover of material, between the top of the tunnel and the bed of the river, can be obtained; and where the channel of the waterway cannot be obstructed during construction. In suitable material very rapid progress can be made, the cast-iron lining facilitating rapid construction. On one tunnel of the Hudson and Manhattan Railroad, driven through Hudson River silt, a length of 72 ft. was built in one day.

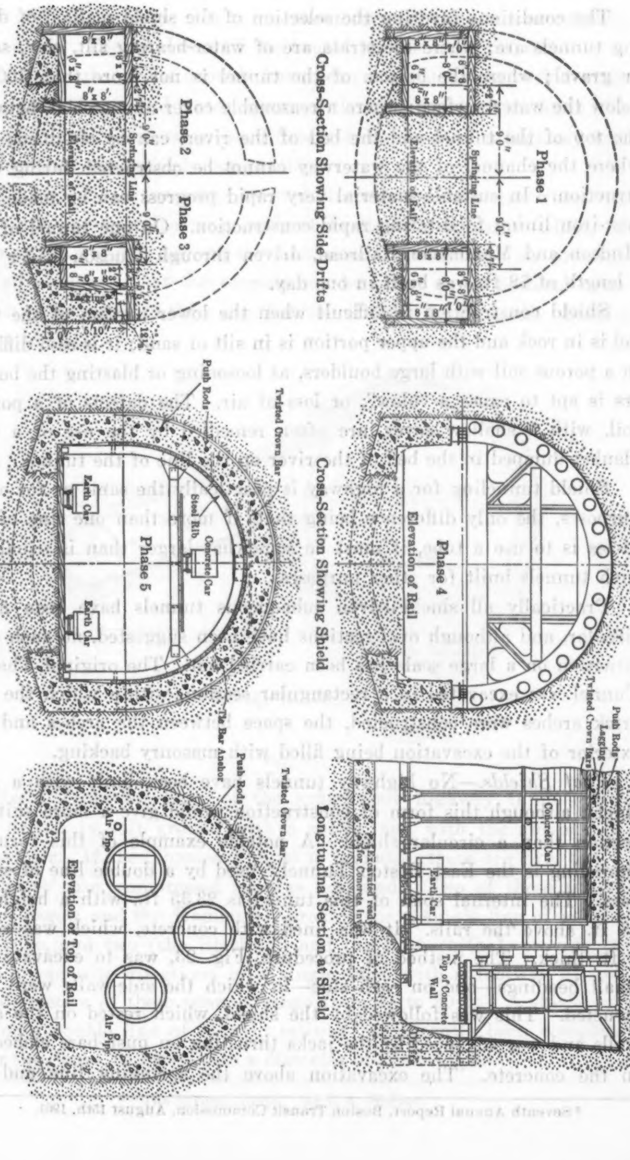
Shield construction is difficult when the lower portion of the tunnel is in rock and the upper portion is in silt or sand; it is also difficult in a porous soil with large boulders, as loosening or blasting the boulders is apt to cause a "blow", or loss of air. The defects of a porous soil, with a shallow cover, are often remedied by the use of a clay blanket dumped in the bed of the river on the line of the tunnel.

Shield tunneling for a highway is practically the same as for other purposes, the only difference being that, if more than one line of vehicles is to use a tube, it must be somewhat larger than is usual for such tunnels built for other purposes.

Practically all shield-driven subaqueous tunnels have been made circular, and although oval sections have been suggested, no such construction on a large scale has been carried out. The original Thames Tunnel was excavated in a rectangular section, within which the two brick arches were constructed, the space between the arches and the exterior of the excavation being filled with masonry backing.

Roof Shields.—No highway tunnels have been built with a roof shield, although this form of construction would give a more suitable section than a circular shield. A notable example of this form of tunneling is the East Boston Tunnel,* used by a double line of street cars. The internal span of this tunnel is 23.33 ft., with a height of 18 ft. above the rails. It was lined with concrete, which was about 3 ft. thick. The method of procedure, Fig. 23, was to excavate two small headings—one on each side—in which the side-walls were constructed. This was followed by the shield, which rested on the side-walls and was advanced by the jacks thrusting on push-bars embedded in the concrete. The excavation above the springing line and the

* Seventh Annual Report, Boston Transit Commission, August 15th, 1901.



EAST BOSTON TUNNEL METHODS OF CONSTRUCTION.

FIG. 23.

concrete arch were under the protection of the shield. The final step was to complete the excavation below the springing line and lay the invert. The material encountered was a stiff blue clay in which there was very little water.

There are probably many places where this form of construction would be suitable, as all that is required is a material stable enough to enable the small advance headings to be constructed and the large face due to the width of the shield to be excavated, this being breast-boarded when necessary.

A variation of this method, which has not been tried, would be to construct the side-walls of concrete and then to build the roof, above the springing line, of cast iron resting on skewbacks, using a roof shield, and finally to complete the section by constructing the invert in concrete. This method might prove economical where the lower part of the tunnel is in rock and the upper part in soft ground, as tunneling with a circular shield is very expensive under these conditions.

Coffer-Dam Methods.—The coffer-dam method was used in the original Chicago River Tunnels. It consisted of enclosing a portion of the river in coffer-dams, pumping out the water, excavating the enclosed space, and then constructing the subaqueous passage, after which the dam was removed and the channel restored; then similar steps were taken on the remaining sections of the river.

The advantages of this method are that the roof of the tunnel can be placed at the minimum depth, thus reducing the gradients of the approaches and the total descent and ascent to be overcome by those using the passage. The fact that it was necessary to lower the Chicago Tunnels would indicate that foresight should be used in fixing the elevations of the roofs of tunnels, built in this way, crossing navigable streams.

Conditions favorable to this form of construction are: the ability to close temporarily part of the river channel at a time, a more or less impervious river bed which will prevent the water from leaking under the coffer-dams and into the space excavated for the tunnels, a stream not subject to excessive floods, etc.

Harlem River Crossing, New York Rapid Transit Subway.—A variation of the coffer-dam method was developed in the Harlem River crossing of the first New York rapid transit subway, which could be used equally well for a highway tunnel. This crossing was constructed

in two sections, one-half the width of the river being closed at one time. The plan of construction was devised by the firm of McMullen and McBean, the contractors, and as the methods used in the south-westerly half of the river differed somewhat from those in the north-easterly half, the former will be described first. In section, the tunnels consist of two cast-iron segments of a circle with a vertical diaphragm separating them, the whole being encased in concrete. (Fig. 24.)

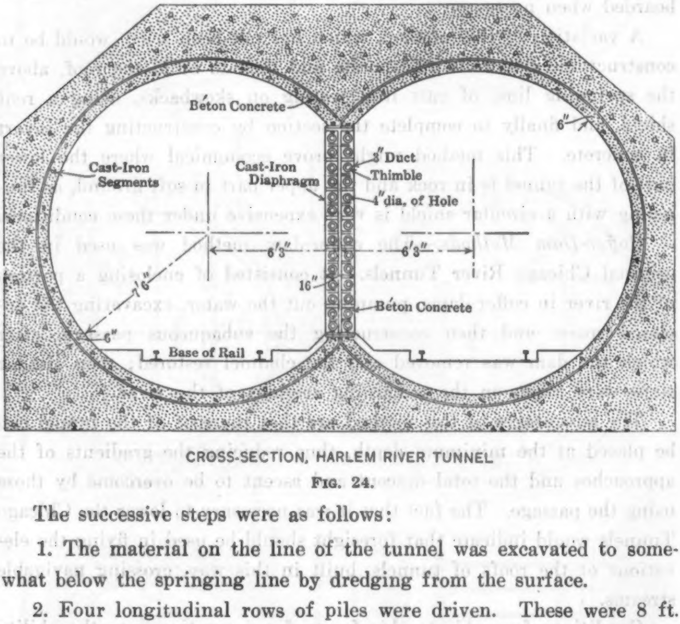


FIG. 24.

The successive steps were as follows:

1. The material on the line of the tunnel was excavated to somewhat below the springing line by dredging from the surface.

2. Four longitudinal rows of piles were driven. These were 8 ft. apart longitudinally and 6 ft. 4 in. transversely. They were then cut off under water so that their tops were about 11 ft. above the axis of the tunnel.

3. The sides of the excavation were then enclosed by sheet-piling, consisting of 12 by 12-in. long-leaf yellow pine timbers, about 65 ft. long, bolted together into units 3 ft. wide. This sheeting was guided by a pile platform along each side and by a timber frame. These piles were then cut off under water by a circular saw.

4. On top of the four rows of bearing piles and the sheet-piles, was then sunk a timber deck consisting of three transverse layers of 12 by 12-in. timbers and two longitudinal layers of 2-in. plank. After this was sunk accurately in place, it was covered with a thickness of about 5 ft. of earth and mud.

5. Several vertical shafts were placed, passing through this deck, and, after placing bulkheads at the ends and air locks, compressed air was turned on and the excavation was completed, the iron lining was erected, and the concrete backing placed. The bearing piles were cut off and removed as the work progressed.

In the construction of the northeasterly section, instead of building a timber air floor resting on sheet-piling, inside which the excavation was completed and the lining placed, the upper half of the lining was erected inside of a floating box and then sunk on the sheet-piling and utilized as an air floor for the construction of the lower half of the work. This floor was built in three sections—two 90 ft. long and one 84 ft. long. When the upper half of the structure had been constructed in the floating box above its final position, the suspender rods were attached, and sufficient water was pumped into the box to sink it. It was then pulled out lengthwise from under the roof, and the latter was sunk into position, the end joint being made by a diver. After being sunk to position, compressed air was installed, the excavation was completed, and the lower half of the lining was constructed. (Fig. 25.)

A design for the application of this method to a highway tunnel was prepared by the Carnegie Steel Company and presented in a paper by R. B. Woodworth, M. Am. Soc. C. E., before the Railway Club of Pittsburgh.* (Fig. 26.) This plan contemplates a roadway 60 ft. wide, with two electric car tracks, the floor and side-walks being of concrete and the roof of steel embedded in concrete.

Another design for a highway tunnel by this system, having two roadways, each 19 ft. 6 in. in width, was shown before the Hoboken Board of Trade on February 4th, 1913. (Fig. 27.)

Detroit Tunnel.—The method used in constructing the Detroit River Tunnel has been described so thoroughly in the paper† by W. S.

* *Official Proceedings*, Railway Club of Pittsburgh, December 22d, 1909.
† *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 288.

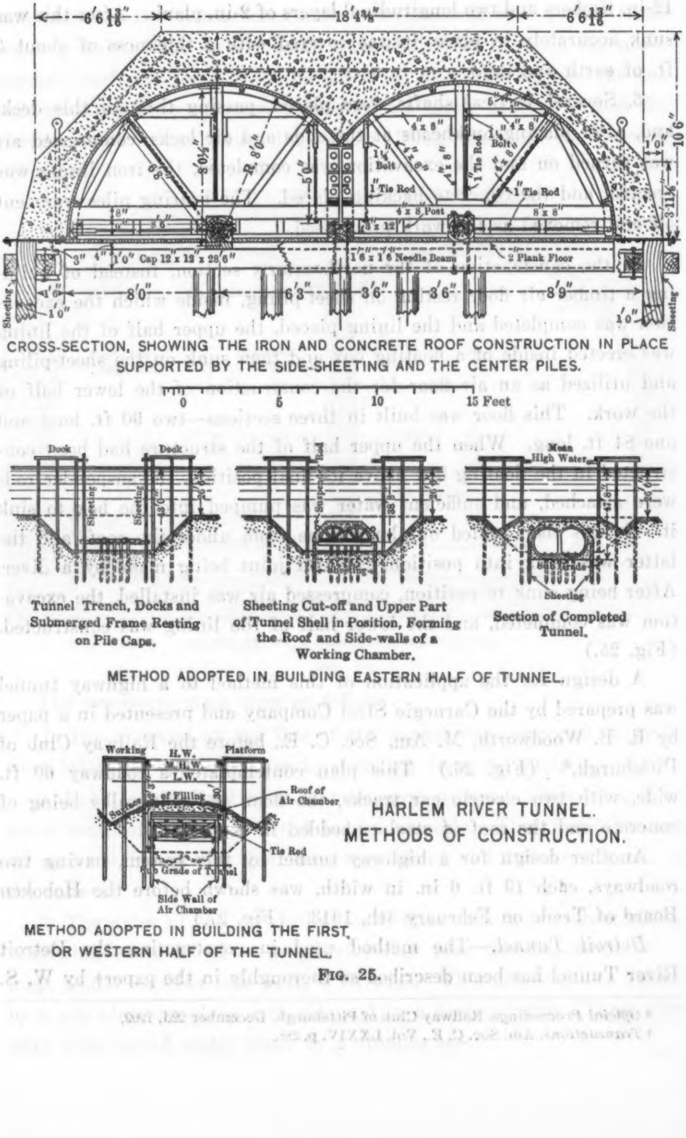
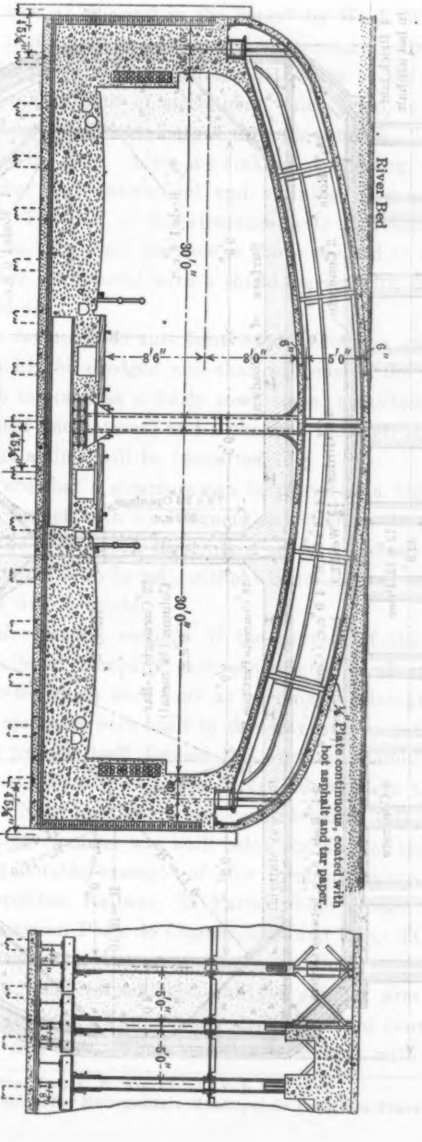

CROSS-SECTION, SHOWING THE IRON AND CONCRETE ROOF CONSTRUCTION IN PLACE SUPPORTED BY THE SIDE-SHEETING AND THE CENTER PILES.
0 5 10 15 Feet
1 Tie Rod
4 x 8 Post
Cap 12 x 12 x 28′6″
1′6 x 1′6 Needle Beam
2 Plank Floor
Sheeting
Dock
Mean High Water
Tunnel Trench, Docks and Submerged Frame Resting on Pile Caps.
Sheeting Cut-off and Upper Part of Tunnel Shell in Position, Forming the Roof and Side-walls of a Working Chamber.
Section of Completed Tunnel.
METHOD ADOPTED IN BUILDING EASTERN HALF OF TUNNEL.
Working
Platform
H.W.
M.H.W.
L.W.
of Filling
Roof of Air Chamber
Tie Rod
Sub Grade of Tunnel
Side Wall of Air Chamber
HARLEM RIVER TUNNEL.
METHODS OF CONSTRUCTION.
METHOD ADOPTED IN BUILDING THE FIRST, OR WESTERN HALF OF THE TUNNEL.

FIG. 25.

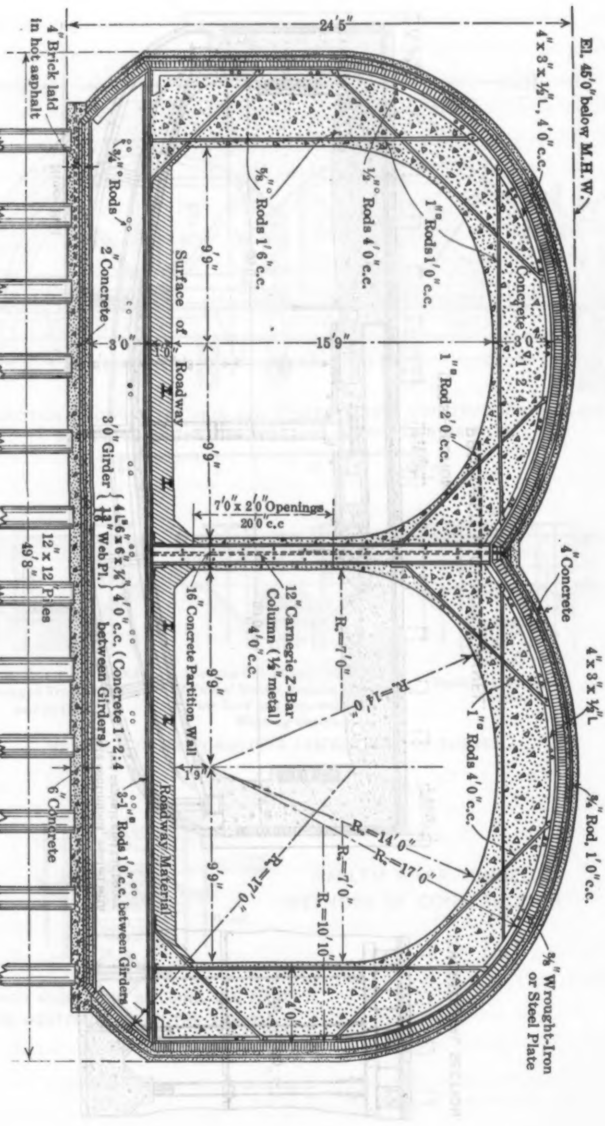


CROSS-SECTION

LONGITUDINAL SECTION

PROPOSED HIGHWAY TUNNEL.

FIG. 26.



CROSS-SECTION, PROPOSED HIGHWAY TUNNEL

Fig. 27.

Kinnear, M. Am. Soc. C. E., and in the paper* by W. J. Wilgus, M. Am. Soc. C. E., that but little comment is necessary. Briefly, the method consisted of dredging a trench along the line of the structure into which were sunk sections of steel tubes which acted as forms to limit the concrete deposited from barges through tremies. These sections were 262 ft. 6 in. long. After the sinking, concreting, and joining of sections, they were unwatered and a lining of concrete was added. (Fig. 28.) The roof of this structure is 41 ft. below the surface of the water, and for some distance is above the bed of the river. The approaches were constructed with a shield, partly with and partly without compressed air.

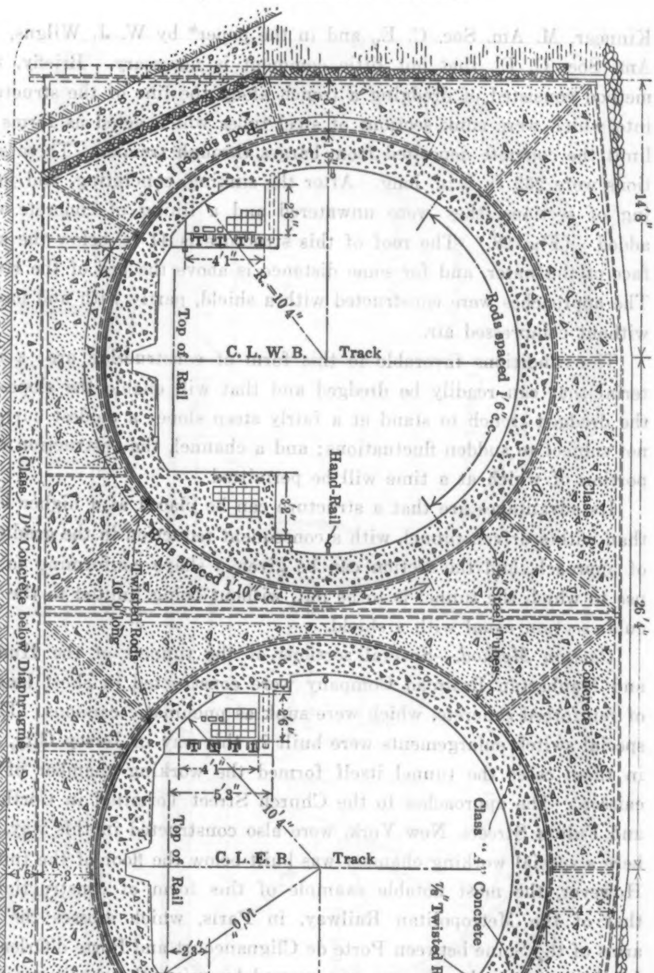
The conditions favorable to this form of construction are: a material that can readily be dredged and that will enable the slopes of the dredged trench to stand at a fairly steep slope; a modest current, not subject to sudden fluctuations; and a channel, the obstruction of a portion of which at a time will be permitted.

Its advantages are that a structure can be placed at a higher level than with a driven tunnel, with a consequent reduction in the gradients of approach, and the section can be given a shape conforming to the use to which the tunnel is to be put, without being limited to the circular section usual with a shield.

Caisson Method.—Several sections of the tunnels of the Hudson and Manhattan Railroad Company were constructed in short lengths of reinforced concrete, which were sunk as pneumatic caissons. Three special switch enlargements were built in this way in Jersey City, and in these cases the tunnel itself formed the working chamber of the caisson. The approaches to the Church Street Terminal in Cortlandt and Fulton Streets, New York, were also constructed in this way, but here a special working chamber was built below the floor of the tunnel. However, the most notable example of this form of construction is that of the Metropolitan Railway, in Paris, which crosses the two arms of the Seine between Porte de Clignancourt and Porte d'Orleans.† In this case the larger arm was crossed by using three caissons, 118, 125.9, and 141.7 ft. long, respectively, and the smaller arm by two caissons, each 65.9 ft. long. Two stations under the land nearby were also constructed in this way. These caissons were sunk with a space

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXV, p. 2.

† "Les Travaux Chemin de Fer Métropolitain Municipal de Paris à la Traversée de la Seine," by L. Biette.



CROSS-SECTION, DETROIT RIVER TUNNEL

FIG. 28.

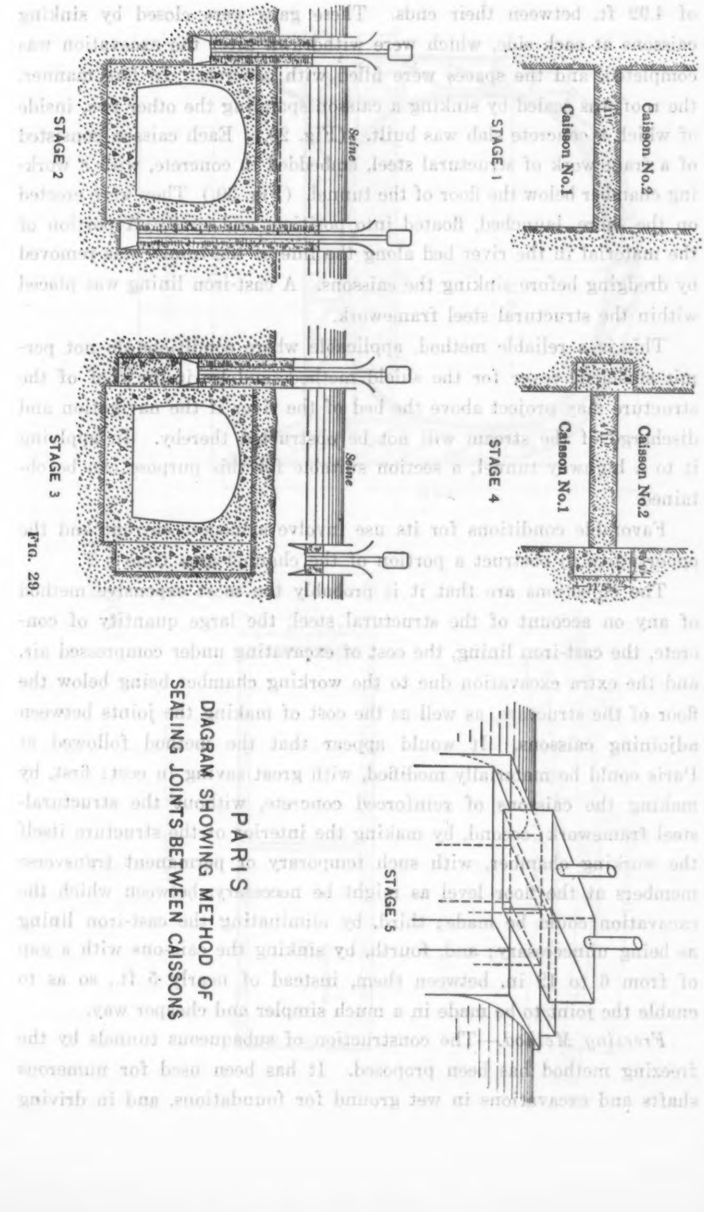


FIG. 29.

PARIS
DIAGRAM SHOWING METHOD OF
SEALING JOINTS BETWEEN CAISSONS

of 4.22 ft. between their ends. These gaps were closed by sinking caissons at each side, which were withdrawn after the excavation was completed, and the spaces were filled with concrete. In this manner, the roof was sealed by sinking a caisson spanning the other two, inside of which a concrete slab was built. (Fig. 29.) Each caisson consisted of a framework of structural steel, embedded in concrete, with a working chamber below the floor of the tunnel. (Fig. 30.) These were erected on the shore, launched, floated into [illegible] portion of the material in the river bed along the line of [illegible] removed by dredging before sinking the caissons. A cast-iron lining was placed within the structural steel framework.

This is a reliable method, applicable where [illegible] not permit [illegible] for the shield method, and where the top of the structure may project above the bed of the river, as the navigation and discharge of the stream will not be obstructed thereby. In applying it to a highway tunnel, a section suitable for this purpose can be obtained.

Favorable conditions for its use involve [illegible] and the [illegible] to construct a portion of the [illegible].

The objections are that it is probably the most expensive method of any on account of the structural steel, the large quantity of concrete, the cast-iron lining, the cost of excavating under compressed air, and the extra excavation due to the working chamber being below the floor of the structure, as well as the cost of making the joints between adjoining caissons. It would appear that the method followed at Paris could be materially modified, with great saving in cost: first, by making the caissons of reinforced concrete, without the structural-steel framework; second, by making the interior of the structure itself the working chamber, with such temporary or permanent transverse members at the floor level as might be necessary, between which the excavation could be made; third, by eliminating the cast-iron lining as being unnecessary; and, fourth, by sinking the caissons with a gap of from 6 to 12 in. between them, instead of nearly 5 ft., so as to enable the joints to be made in a much simpler and cheaper way.

Freezing Method.—The construction of subaqueous tunnels by the freezing method has been proposed. It has been used for numerous shafts and excavations in wet ground for foundations, and in driving

of 4.92 ft. between their ends. These gaps were closed by sinking caissons at each side, which were withdrawn after the excavation was completed, and the spaces were filled with concrete. In like manner, the roof was sealed by sinking a caisson spanning the other two, inside of which a concrete slab was built. (Fig. 29.) Each caisson consisted of a framework of structural steel, embedded in concrete, with a working chamber below the floor of the tunnel. (Fig. 30.) They were erected on the shore, launched, floated into position, and sunk. A portion of the material in the river bed along the line of the tunnel was removed by dredging before sinking the caissons. A cast-iron lining was placed within the structural steel framework.

This is a reliable method, applicable where conditions do not permit sufficient cover for the shield method, and by it the roof of the structure may project above the bed of the river if the navigation and discharge of the stream will not be obstructed thereby. In applying it to a highway tunnel, a section suitable for this purpose can be obtained.

Favorable conditions for its use involve a modest current and the opportunity to obstruct a portion of the channel at a time.

The objections are that it is probably the most expensive method of any on account of the structural steel, the large quantity of concrete, the cast-iron lining, the cost of excavating under compressed air, and the extra excavation due to the working chamber being below the floor of the structure, as well as the cost of making the joints between adjoining caissons. It would appear that the method followed at Paris could be materially modified, with great saving in cost: first, by making the caissons of reinforced concrete, without the structural-steel framework; second, by making the interior of the structure itself the working chamber, with such temporary or permanent transverse members at the floor level as might be necessary, between which the excavation could be made; third, by eliminating the cast-iron lining as being unnecessary; and, fourth, by sinking the caissons with a gap of from 6 to 12 in. between them, instead of nearly 5 ft., so as to enable the joint to be made in a much simpler and cheaper way.

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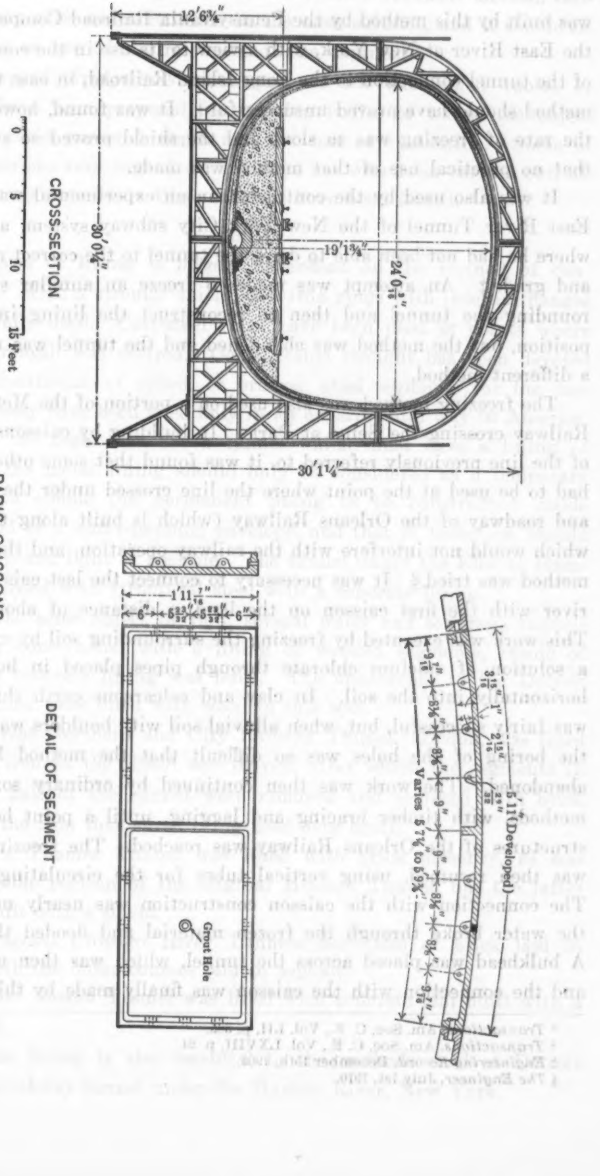


PARIS CAISSONS

FIG. 30.

a tunnel in wet ground in Stockholm.* A short experimental tunnel was built by this method by the Pennsylvania Railroad Company under the East River at New York, with a view to its use in the construction of the tunnel connection to the Long Island Railroad, in case the shield method should have proved unsuccessful.† It was found, however, that the rate of freezing was so slow, and the shield proved so successful, that no practical use of that method was made.

It was also used by the contractor, in an experimental way, on the East River Tunnel of the New York City subway system, at a point where he had not been able to drive the tunnel to the correct alignment and grade.‡ An attempt was made to freeze an annular space surrounding the tunnel and then to reconstruct the lining in its true position, but the method was abandoned and the tunnel was rebuilt by a different method.

The freezing method was also used on a portion of the Metropolitan Railway crossing the Seine at Paris. In building by caissons the part of the line previously referred to, it was found that some other method had to be used at the point where the line crossed under the structure and roadway of the Orleans Railway (which is built along the quay), which would not interfere with the railway operation, and the freezing method was tried.§ It was necessary to connect the last caisson in the river with the first caisson on the land, a distance of about 200 ft. This work was executed by freezing the surrounding soil by circulating a solution of calcium chlorate through pipes placed in holes bored horizontally into the soil. In clay and calcareous earth this method was fairly successful, but, when alluvial soil with boulders was reached, the boring of the holes was so difficult that the method had to be abandoned. The work was then continued by ordinary soft ground methods, with timber bracing and lagging, until a point beyond the structures of the Orleans Railway was reached. The freezing process was then resumed, using vertical tubes for the circulating medium. The connection with the caisson construction was nearly made when the water broke through the frozen material and flooded the tunnel. A bulkhead was placed across the tunnel, which was then unwatered, and the connection with the caisson was finally made by this method.

* *Transactions*, Am. Soc. C. E., Vol. LII, p. 365.
† *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 24.
‡ *Engineering Record*, December 15th, 1906.
§ *The Engineer*, July 1st, 1910.

Notwithstanding the general lack of success with this method, conditions may arise where it would be the most suitable, and it should be considered as one of the resources of the engineer.

Submerged Bridges.—Various projects for submerged tubes, or bridges, have been proposed for crossing bodies of water, the object being to avoid excessive depth and the cost of excavation. Some of these plans are very ingenious, but none has ever been carried out, although many patents on these schemes have been issued.

LINING.

The type of lining is largely dependent on the method of construction. With a circular shield, cast-iron rings with internal flanges are generally used. Cast-steel rings have been used at points where special strength was desired. Considerable thought has been devoted to the substitution of rolled, or pressed, steel segments for the cast iron, but they have been tried only in an experimental way in America. The idea has been that, for a tunnel which must have a lining of masonry, the metal lining should only be considered as a temporary sheeting to enable the permanent lining to be constructed inside thereof and as a water-proofing envelope, and that it, therefore, should be as cheap and light as possible. The tunnel under the Elbe at Hamburg was lined with structural steel with a concrete lining.

With the same object in view, several water and sewerage tunnels have been constructed with a lining of wooden segments, inside of which the concrete lining was built. The approaches to the Detroit River Tunnel were lined with wood in this way.

The Great Northern and City Railway Tunnel, London, was lined with cast iron as the shield was driven, after which the segments in the lower half of the tunnel were removed and replaced with brick masonry; the iron thus saved being used over again.

Brunel's Thames Tunnel was lined with brick masonry, as was a considerable portion of the original Hudson Tunnel, but the latter was not built with a shield.

The original Chicago River Tunnels were also of brick laid in cement, with the outer courses laid in asphalt.

The East Boston Tunnel was lined with concrete and built with a roof shield.

Concrete lining is also essential to the methods used at Detroit and in the subway tunnel under the Harlem River, New York.

Concrete blocks, in lieu of cast-iron lining, for shield-driven tunnels have been proposed and patented by John F. O'Rourke, M. Am. Soc. C. E. (Fig. 31.)*

In a favorable stratum, a subaqueous highway tunnel might be built without any artificial lining at all, but, for sanitary reasons and for ease in illumination, it is doubtful if this would be advisable. The East River Gas Tunnel, at Seventy-first Street, New York City, is about 100 ft. below the surface of the water and is only lined for a short distance in the vicinity of fissures in the rock.

As to the relative advantages of the different linings, cast iron is especially suited to shield construction, and has the advantage of being readily made water-tight, together with its ability to resist the effects of corrosion. For a highway tunnel, it has the disadvantage of being unsuited to any but a circular section. Concrete is readily adapted to a section of any shape. When used with a shield, it must have metal members embedded in it to take the thrust of the shield jacks. When in water-bearing strata under great head, it is difficult to make it entirely water-proof in itself, and it is also difficult to apply an extraneous envelope of water-proofing. A warning should also be sounded in regard to its use in strata containing acid or alkali waters, which rapidly disintegrate concrete. Such waters, though not usual, are not rare. In the first attempt to build the Detroit River Tunnel, sulphur water and sulphur gas were found in large quantities, and sulphuric acid has been encountered on other underground works, so that analyses and tests of underground waters should be made on all tunnel work.

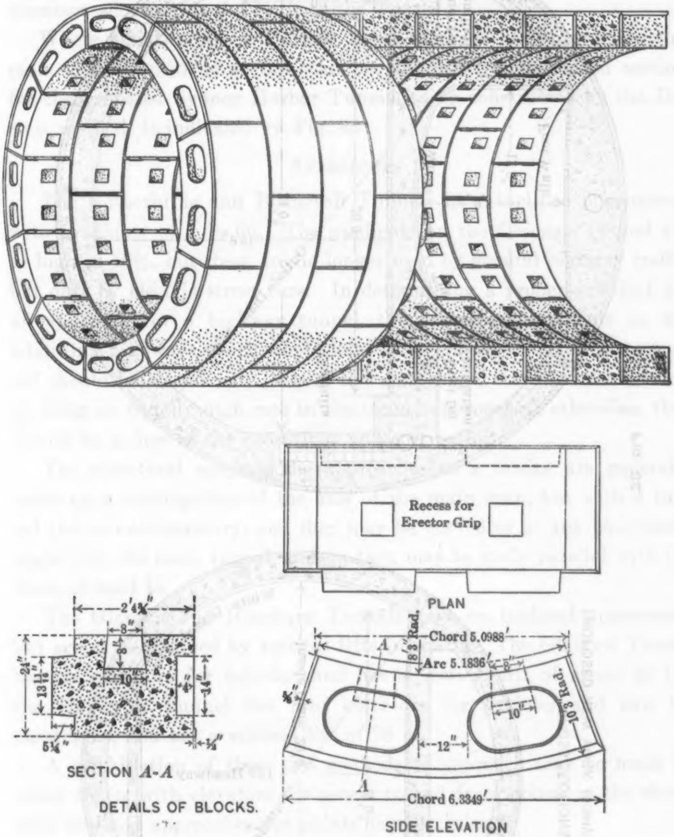
An electrical survey should also be made, in order to determine the probable effect of such a tunnel in forming a new path for stray electric currents, and steps should be taken to prevent damage to the structure from electrolysis.

SECTION.

The shape of the section is largely determined by the method of construction, rather than by the use to which the tunnel is to be put. Shield tunnels have generally been circular, and this section is not generally well adapted to highway purposes, a semicircular, or horseshoe section being more suitable. The Blackwall Tunnel has a 16-ft. roadway and two 3 ft. 1½-in. footwalks, and the Rotherhithe Tunnel

* *Engineering News*, November 28th, 1912.

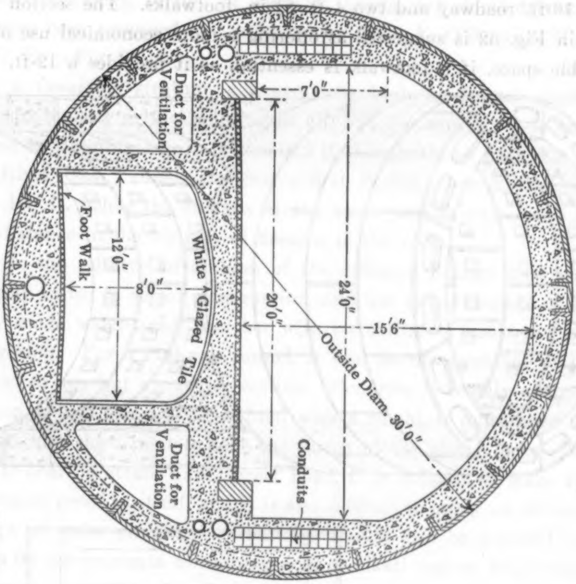
has a 16-ft. roadway and two 4 ft. 8½-in. footwalks. The section indicated in Fig. 32 is suggested as offering a more economical use of the available space, if a footwalk is essential, as it provides a 12-ft. foot-



CONCRETE TUNNEL LINING

FIG. 31.

walk and 20-ft. roadway. In tunnels more than 1 mile in length, it is doubtful if a sufficient number of persons would use the footwalk to warrant its construction, particularly where facilities are provided for crossing in electric cars in the same, or another, tunnel. If the



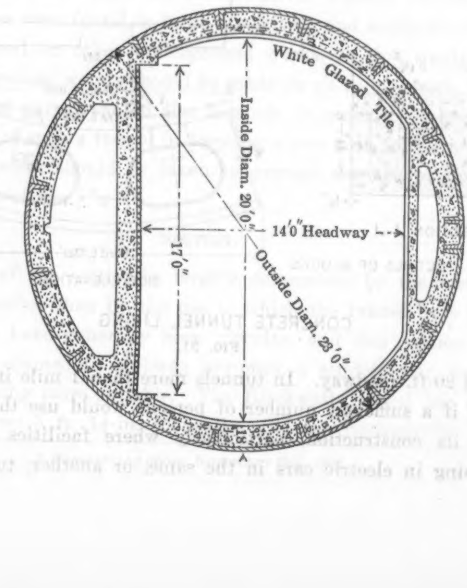


FIG. 32.

CROSS-SECTIONS, PROPOSED NEW YORK HIGHWAY TUNNEL

tunnel is made of sufficient size for only the roadway, a material reduction in area can be made, as a 17-ft. roadway can be obtained in a tunnel 23 ft. in diameter, as shown by Fig. 32, instead of the 30-ft. diameter adopted for the Rotherhithe Tunnel.

Where the Detroit or Harlem River methods of construction are used, a more suitable section can be obtained. A suggested section for the proposed Sydney Harbor Tunnel, to be constructed by the Detroit method, is indicated by Fig. 33.

Approaches.

The Rotherhithe and Blackwall Tunnels have inclined approaches, with gradients of 1 in 36. The gradients on the Chicago Tunnel are as high as 10%, but these are no longer used by general highway traffic, but only by electric street cars. In determining a proper gradient for an approach for a highway tunnel, the prevailing gradients on the adjoining highways which would have to be traversed to reach the tunnel should be considered. Where these are high, there is no objection to using an equally high rate in the tunnel approaches; otherwise, they should be as low as the conditions and cost permit.

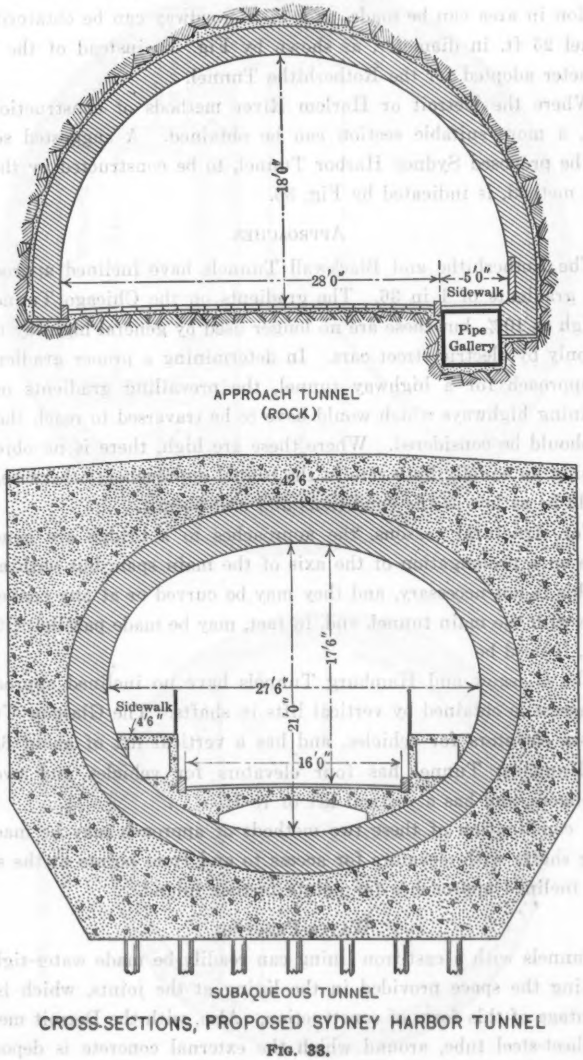
For structural reasons, the approaches to a bridge are generally made on a prolongation of the axis of the main span, but with a tunnel this is not necessary, and they may be curved or at any convenient angle with the main tunnel, and, in fact, may be made parallel with the river, if need be.

The Glasgow and Hamburg Tunnels have no inclined approaches, but access is obtained by vertical lifts in shafts. The Glasgow Tunnel has six elevators for vehicles, and has a vertical lift of about 65 ft.; the Hamburg Tunnel has four elevators for vehicles and two for passengers, and has a vertical lift of 78 ft.

A combination of these two methods of approach may be made by using shafts with elevators for access to and from points at the shore, with inclined approaches for points farther inland.

Water-Proofing.

Tunnels with a cast-iron lining can readily be made water-tight by caulking the space provided in the lining at the joints, which is one advantage of this form of construction. Also, with the Detroit method, the sheet-steel tube, around which the external concrete is deposited, can be caulked and will keep the tunnels tight. Tunnels sunk by the



CROSS-SECTIONS, PROPOSED SYDNEY HARBOR TUNNEL

FIG. 38.

caisson process can have an inner or outer shell which can be caulked and will serve as a water-tight envelope.

Masonry under-river crossings, built by the open coffer-dam method, can be water-proofed by the membranous method, using felt, duck, canvas, or burlap, and pitch or asphaltum, or an enveloping course of brick masonry laid in asphaltic or bituminous mastic.

With a concrete lining laid in a driven tunnel, satisfactory water-proofing is a more difficult problem. If the tunnel is constructed with the aid of compressed air, the application of a bituminous compound that must be heated to be applied, is difficult.

In the approaches to the Detroit River Tunnel, which were lined with concrete, the water-proofing consisted of from 3 to 11 plies of felt and pitch applied to the wooden lagging before the concrete was placed.

The East Boston Tunnel was constructed in a more or less impervious clay, and no water-proofing was used.

The liberal use of grout at a greater pressure than that due to the hydrostatic head, will add greatly to the imperviousness of the concrete and surrounding material.

In the reconstruction of the La Salle Street Tunnel, in Chicago, a water-proofing compound was added to the concrete with a view to making it water-tight.

Some concrete-lined electric railway tunnels have been water-proofed with an internal plaster coat of cement to which water-proofing compound had been added, but, owing to contraction cracks, it is difficult to get an absolutely water-tight structure by this method.

VENTILATION.

Thus far the ventilation of highway tunnels has not proved to be a serious problem. In the Blackwall Tunnel ventilation ducts and provision for fans were made, but the fans have never been found necessary. In the Rotherhithe Tunnel no provision for ventilation has been made. The traffic in this tunnel has been as follows:

	1911.	1912.
Total number of vehicles, including motors	896 629	973 336
Motor vehicles only	21 008	26 998
Maximum number of vehicles passing through the tunnel in one hour...		325

Of the latter, 10 were motor-driven.

The figures for the Blackwall Tunnel are somewhat similar.

Although, under the foregoing conditions, artificial ventilation has not proved necessary, it is probable that, with the increasing use of motor-driven vehicles having internal-combustion engines, provision for the mechanical ventilation of long highway tunnels will have to be made.

The exhaust gases given off by a gasolene motor consist of carbon monoxide, carbon dioxide, and free oxygen. Of these, the most poisonous is the carbon monoxide, and if sufficient fresh air is provided to dilute this to within safe limits, the other gases will not cause any trouble. In addition to this, sufficient air must be provided to furnish the oxygen necessary for combustion. The character and quality of these gases has been determined very accurately in some recent tests* which indicate that, with a 6-cylinder, 48-h.p. motor, the quantity of air required to dilute the gases given off, within safe limits, would be about 8 000 cu. ft. per sec. when in first gear, 4 000 cu. ft. per sec., when on second gear, and 2 000 cu. ft. per sec., when on third gear. Based on these figures, it will be found that the ventilation of such a tunnel, even assuming that motor cars follow each other as closely as safety permits, can be readily accomplished. It can be effected by withdrawing air from the center of the tunnel through conduits under the floor, as in the Blackwall Tunnel, or above the clearance line, as in the East Boston Tunnel, or through separate ventilating passages, as in the Mersey Tunnel.

DRAINAGE.

The Severn and Mersey Tunnels are both drained by letting the water leaking into them flow in separate tunnels, constructed for the purpose, from the lowest points in the main tunnels to shafts on the shore, where it is removed by pumping. In both these cases the tunnels were constructed in rock and lined with brick masonry, and the leakage was very large, but in more recent tunnels, lined with iron, steel, or concrete, the leakage is so small† that such special drainage headings are hardly necessary. In these latter tunnels the water is removed by pumping from the sumps through pipes laid along the sides of the tunnel to the shore.

* "A Comprehensive Motor Test," by Herbert Chase, *Transactions*, Soc. of Automobile Engineers, 1912, p. 40.

† Detroit Tunnel, *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 349; Hudson Tunnel, *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI, p. 169.

The art of boring large-sized holes in any direction has developed to such an extent that there is no reason why a hole cannot be bored from the sump upward through the overlying material to the bed of the river through which to discharge the drainage and thus save the long length of pipe to shore and the increased friction head.

Conclusions.

In this paper the writer has endeavored to assemble the published facts relative to all the subaqueous highway tunnels in the world, as well as of subaqueous tunnels built for other purposes, where the methods of design or construction would have an application for highway purposes. The paper, therefore, is largely a compilation, the information having been obtained from the *Transactions* of this and other engineering societies, and from the technical press. Credit for this information is generally given, but in a number of cases where the information came from various sources, the authority is not stated, and the writer wishes to take this opportunity to express his indebtedness to those who have written on this subject.

The need for improved methods of vehicular communication around harbors is manifest, and it would seem that the furnishing of this by highway tunnels has not received the attention it should. This paper, therefore, has been prepared in the hope that it will develop a discussion which will stimulate attention to tunneling for this purpose.

Commercial transportation was formerly largely conducted on the highways, and the world will shortly complete a century of railway transportation, one of the consequences of which was neglect of the highways, but the present tendency is to return to the highway for short-haul transportation, largely owing to the development of the motor vehicle for pleasure and business, which is leading to a revival in the study of road construction to meet new conditions, and highway tunnels, in many localities, are a necessary link to make this system of communication continuous.

DISCUSSION

Mr. Davies. J. V. DAVIES,* M. AM. SOC. C. E.—In working out the study of any individual case that is presented, the thing that impresses the speaker more than anything else, is the great scope for the display of engineering ability in the various special lines of the Profession. In the last few years, in connection with the Interstate Bridge and Tunnel Commission, the speaker has discussed at public hearings the relative advantages of bridges or tunnels for vehicular traffic across the Hudson River to such an extent that the subject has become almost tiresome.

There are a great many marked advantages in certain particular cases in the use of tunnels for subaqueous highway purposes over the use and adaptation of the bridge: they are better protected; they present less external obstruction; there is less cost of maintenance, which is a very large item; and per track or per roadway, they will probably cost materially less. Another obvious advantage is that a bridge has to be constructed in the first instance for the full permanent use and purpose for which it is required; whereas, in the case of tunnels, single roadways can be constructed and added to as the volume of business extends, and such additional tunnels can be located and distributed as traffic requires.

It is certain that there will be a growing use of tunnels for highway purposes. The speaker, while in London recently, was rather interested in the fact that the Rotherhithe and Blackwall Tunnels had been closed entirely to traffic, due to the fear of war conditions, involving the use of bombs or explosives which might cause serious injury to those structures. Reference has been made to a pertinent question, arising from the conditions developed during the procedure of war in Europe, as to the destruction of bridges across rivers by the contending armies, and it is really a matter of thought, whether a tunnel in such cases—with the ability to flood it and render it temporarily useless without permanent injury—has not a very great advantage over a bridge for ordinary highway purposes.

On the question of the transporter bridge the speaker has studied very closely three of the largest of such bridges—those at Runcorn and Newport, in England, and the one at Rouen, France. At all these bridges there is considerable rise and fall of the tide. At Newport, particularly, the average rise and fall of the tide in the River Usk, where the bridge is located, is some 35 ft. When the water is at high level the river is a very fine and noble stream. At low water there is a mud valley with a small creek at the bottom of it. A ferry service is impossible. An ordinary highway bridge would be difficult and expensive to build, and even with draw-spans

* New York City.

would be a serious interference to navigation. A tunnel, on the other hand, on account of the banks and river valley, would require such approaches as would render it undesirable for such purpose; and where the traffic is very small, as it is in this case, and as it is also in Rouen and Runcorn, there is no warrant for constructing any very large and expensive draw-bridge, or, on the other hand, any expensive tunnel, for such conditions; whereas, the transporter bridge, giving a fixed end landing and reasonably frequent service, at low initial cost, serves the purpose of transportation admirably. This is an illustration of the fact that each individual case must be considered on its own merits. Mr. Davies.

The greatest point of advantage in the use of a subaqueous tunnel, over any other method of conveyance for highway uses, is its flexibility of location. The Rotherhithe Tunnel does not go squarely across the river; even the approaches are at varying angles; and the location and design of the tunnel were evolved to connect two great populous districts which had business intercourse and business interests on each side of the Thames, where that intercourse was urgently necessary. The tunnel, therefore, was constructed on a zigzag alignment. Crossing the Thames on a diagonal line, it allowed communication to be had between districts that required that communication, and, to the speaker's mind, more than anything else, that flexibility of location for highway purposes is one of the most important features in the consideration of this subject.

DUNCAN D. MCBEAN,* ESQ.—Mr. Snyder gives a cross-section of the Harlem River Tunnel (Fig. 24), and sections showing the working platforms (Fig. 25), which, for convenience, were used instead of scows in constructing the subaqueous working chamber in which the tunnel was built. In discussing the method of constructing the tunnel, Mr. Snyder writes of it as being "a variation of the coffer-dam method", which is not only a misnomer, but also misleading. The patent obtained by the writer for this method is for a "Subaqueous Working Chamber"; he believes, therefore, that the method should be designated the Subaqueous Working Chamber method. Mr. McBean.

The advantages of this method are not only those accredited to the Detroit River method by Mr. Snyder, that is:

"That a structure can be placed at a higher level than with a driven tunnel, with a consequent reduction in the gradients of approach, and the section can be given a shape conforming to the use to which the tunnel is to be put, without being limited to the circular section usual with a shield";

but it also has many other advantages, some of which are as follows:

In cases where the circular form is desirable, instead of having a metal shell completely encircling each tube, it permits of joining

* New York City.

Mr. McBean. the upper and lower segments of the metal shells to a vertical partition wall between them, thereby forming a twin-tube tunnel, and very materially reducing the quantity of materials required and likewise greatly reducing the surface of metal shells exposed to hydrostatic pressure; it enables the excavation to be made to the neat width and depth required for the structure; and it permits of placing all the masonry, both outside and inside the metal shells, in air.

The water-proofing can be placed on the outside of the masonry, thereby permitting the utilization of all the structural masonry to withstand the surrounding hydrostatic pressure, and thereby greatly diminishing the quantity of material necessary to overcome the buoyancy of the structure during construction.

Mr. Snyder shows a cross-section of the Detroit River Tunnel (Fig. 28), but it is submitted that his conclusion, that

"The method used in constructing the Detroit River Tunnel has been described so thoroughly in the paper by W. S. Kinnear, M. Am. Soc. C. E., and in the paper by W. J. Wilgus, M. Am. Soc. C. E., that but little comment is necessary,"

finds scant support in the papers* referred to.

An analytic examination of the several papers describing the construction of the Detroit River Tunnel shows: That sections of concrete forms consisting of the metal shells for inner forms, with vertical walls of sheeting attached thereto for outside forms (which, taken together, may properly be called sectional skeleton frames of a subaqueous working chamber), were lowered into the dredged trench and rested on temporary supports under the ends of the sections.† These temporary supports consisted of grillages supported by piles, or as Mr. Kinnear calls them,‡ "a 10 by 10-in. timber leg, or spud, about 10 ft. long, which was driven into the clay by an ordinary pile-driver drop-hammer * * *". Under the skeleton frame when resting on its temporary supports, the distance to the bottom of the trench varied "from 1 to 6 ft., according to the depth of trench left by the dredge."§ The foundation concrete under the skeleton frame was deposited through tremie pipes in the water on the muddy bottom of the trench. In order to use tremie pipes for depositing the concrete in the water under the structure, it was necessary to place the tubes 3 ft. apart and to encircle each tube completely with a metal shell, thereby not only causing the track centers in the completed tunnel to be 26 ft. 4 in. apart (in the Harlem River Tunnel the distance between the track centers is only 12 ft. 6 in.), but also increasing the buoyancy, to overcome which necessitated the placing of a large

* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 288; and *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXV, p. 2.

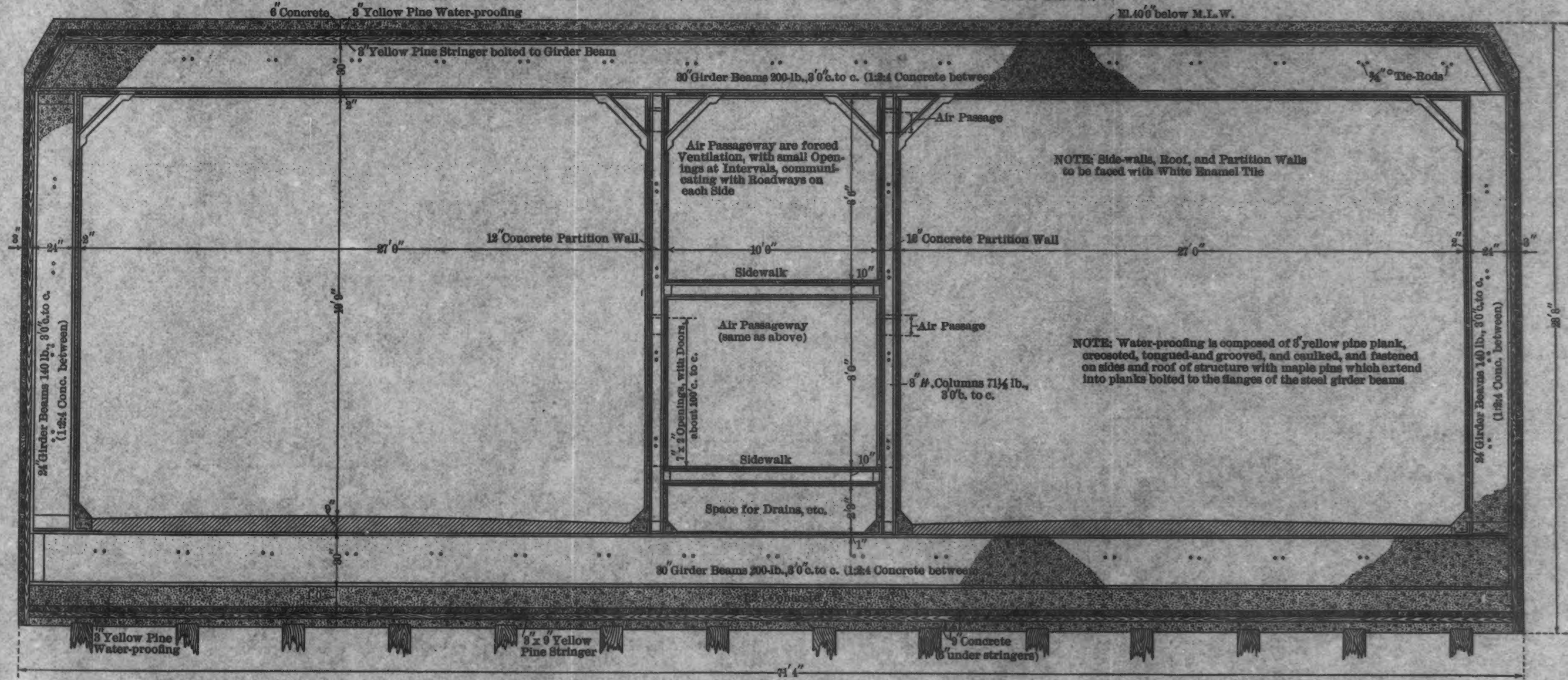
† *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 330.

‡ *Ibid.*, p. 331.

§ *Ibid.*, p. 338.

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1316.
McBEAN ON
SUBAQUEOUS HIGHWAY TUNNELS.

DESIGN FOR ROADWAY TUNNEL UNDER DETROIT RIVER,
BETWEEN DETROIT, MICHIGAN AND WINDSOR, ONTARIO,
IN COMMEMORATION OF 100 YEARS PEACE BETWEEN UNITED STATES AND GREAT BRITAIN



surplus of concrete on the outside of the metal shells. The skeleton frame around the outside of the metal shells was then filled with concrete through tremies, thereby completing the side-walls and roof of a subaqueous working chamber. The water within the metal shells was then pumped out, resulting in the completion of a pneumatic subaqueous working chamber, within which the "tunnel proper" was built.* Mr. McBean.

Mr. Kinnear states that in the fall of 1904, a committee was appointed to report on the feasibility of a tunnel under the Detroit River, at Detroit, and that the report was to include estimates of cost, etc. He states that the committee made the necessary investigations, reported favorably on the construction of a tunnel, and submitted estimates covering the different gradients for approaches, recommending a 2% grade west bound and a 1½% grade east bound. That the estimate as then prepared for the recommended tunnel was a good one, has been demonstrated by the actual cost of construction.

He likewise informs us that in July, 1905, the Committee was superseded by an Advisory Board of Engineers, and that the work of preparing plans was started in August, 1905, but he fails to mention the source of the Committee's data on which it based its estimate of the cost for the construction of the tunnel by the Subaqueous Working Chamber method, concerning which Mr. Hoff writes:

"effected a saving of about \$2 000 000 over a shield-driven tunnel with compressed air, and to this should be added the capitalized saving in annual cost of operation on account of the tunnel being placed some 15 ft. higher, thus reducing by this amount the vertical lift of the tonnage passing through."†

Mr. Snyder's Fig. 27 is a cross-section of a roadway tunnel, which the writer prepared and submitted to the Hoboken Board of Trade. Since then he has prepared and submitted a design (Fig. 34) for a roadway tunnel under the Hudson River, on the proposed Canal Street Line, accompanied with a proposal to build the tunnel and its approaches for the sum of \$5 500 000—just one-half the estimated cost of building a circular tunnel with roadways of the same width by the shield method. The writer has also prepared and submitted a design (Plate VIII) for a proposed highway tunnel under the Detroit River, at Detroit, Mich. The number of piles under the latter structure is greater than would be required ordinarily, except in places where the bottom of the river is filled in for the purpose of supporting the tunnel.

It was the writer's conception to build subaqueous tunnels, according to these designs, by constructing long sections of the tunnels in pontoons in which all the structural material would be put in place,

* *Ibid.*, pp. 303, 328, 362.

† *Ibid.*, p. 371.

Mr. McBean.

DESIGN FOR ROADWAY TUNNEL UNDER RIVERS

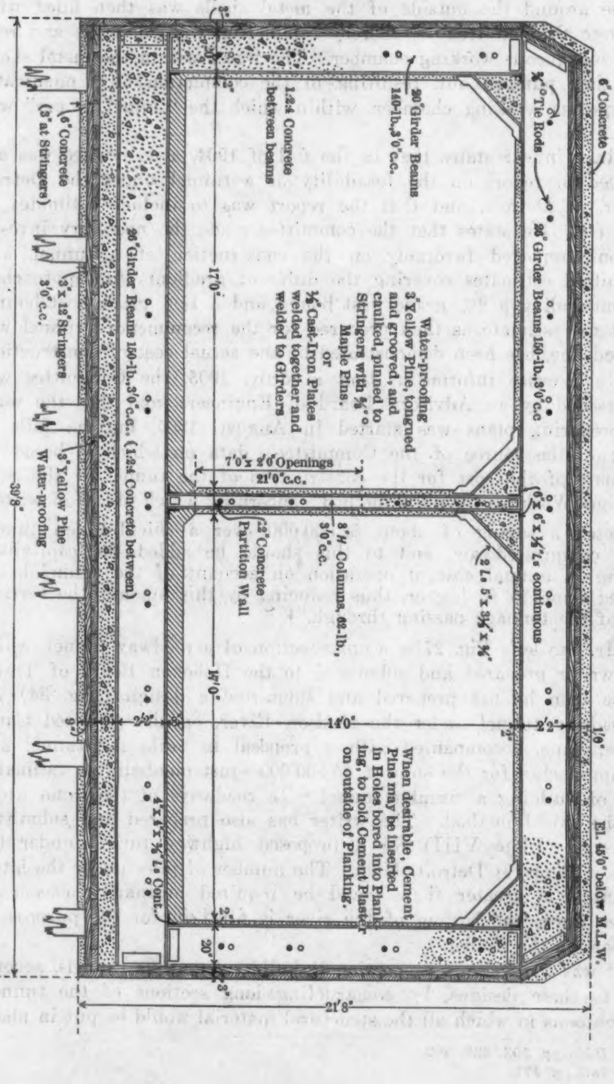


FIG. 34.

except the concrete between the bottom girders inside of the side-walls; bulkheading the ends of the sections; attaching either to the top or to the underside of the bottom girders a temporary water-tight floor; sinking the pontoon and removing it from under the structure; overcoming the buoyancy of the structure by placing within it on its temporary floor a part of the material required for the bottom concrete; then lowering the structure on a temporary foundation of piles, driven into the bottom of the dredged trench, and cut off and capped at the proper depth to support some of the bottom girders; admitting a pressure of air, equal to the hydrostatic pressure at the level of the temporary floor; and then removing the temporary floor. The structure so constructed would then form a working chamber. The foundation would then be cleaned out, the foundation concrete, water-proofed with planking, would be put in place in air, and the section of tunnel completed. Mr. McBean.

The rectangular form of these designs, together with the water-proofing of creosoted, tongued-and-grooved, yellow pine planking, pinned on the outside of the structural material, renders them simple, durable, and economical for future subaqueous tunnel construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1317

THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.*

BY F. LAVIS, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. PHILIP W. HENRY, JAMES J. HILL, T. A. CORRY, FRANK FOSTER, H. DEANE, G. F. F. OSBORNE, JAMES ALEXANDER SMITH, AND F. LAVIS.

SYNOPSIS.

The question of the gauge of railways, though not important in the United States, where it was solved many years ago by the adoption of 4 ft. 8½ in. as a standard, is a live issue in South and Central America, where there are lines of nine different gauges, those most in use being 1 meter, 4 ft. 8½ in., and 5 ft. 6 in. During a recent visit to the Argentine the writer made an investigation of this subject, some of the results of which are embodied in this paper.

The railways of South America have been most largely developed, thus far, by European capital and engineers, but, owing to the changed conditions which have placed, or will soon place, the United States in the position of an importer of food supplies and a seeker for markets for its manufactured products, it seems evident that we must take a much greater and more intimate interest in South and Central American affairs. This question of gauge, therefore, is one which may interest many American engineers in the not far distant future.

* Presented at the meeting of April 1st, 1914.

The subject is treated in the order and under the several different headings mentioned below, the conclusions being briefly summarized as follows:

Curvature.—Within the limits of the practical operation of railroads, any radius curvature which is feasible on meter gauge is as practical on 4 ft. 8½-in. or even 5 ft. 6-in.

Gradients.—The working of trains on steep gradients is not affected by gauge, except in so far as the narrow-gauge cuts down the locomotive capacity and is, therefore, at a great disadvantage.

Location.—In view of the above, the narrow-gauge (3 ft. 0-in. to 3 ft. 6-in.) does not permit better adaptability of the alignment to the conformation of the ground.

Cost of Construction.—This is so little more for 4 ft. 8½-in. than for 3 ft. 0-in., or 3 ft. 6-in., as to be more than counterbalanced by economy in operation.

Speed.—This is an important feature in railway operation, which is adversely affected by narrow gauge.

Stability.—The relative stability of narrow-gauge trains in motion is much less than that of trains on medium- or broad-gauge, and, consequently, high speeds are impractical on narrow-gauge, and may be dangerous.

Track Stresses.—Much higher track stresses are produced on the narrow-gauge. There is greater impact, and therefore more difficulty in maintaining track on narrow- than on broad-gauge.

Rolling Stock—Cars.—The greater width in proportion to the gauge of narrow-gauge cars is considered a necessary evil rather than an advantage, and it is shown that there is probably no advantage to be gained by making the broad (5 ft. 6-in.) gauge cars any wider than they are at present. In the matter of passenger cars, the narrow-gauge is at a decided disadvantage both as to cost per unit of accommodation and in dead weight per passenger. In freight cars there is little difference in cost within the limits of size possible on the narrow-gauge, though cars of twice the capacity are available for medium- or broad-gauge. The increase in weight of modern freight trains, in order to decrease the cost of operation has made heavier draft rigging necessary, thus tending to increase the proportion of dead to paying load to a much greater extent on cars of small capacity than on the larger ones, than has been the case heretofore.

Locomotives.—The cost per unit of capacity of locomotives is about the same for each gauge, up to the limit of the narrow-gauge, which limit for all types is about half that of the 4 ft. 8½-in. or 5 ft. 6-in. There is an economy in the use of heavier types of locomotives not available for the narrow-gauge, and the use of cars of large capacity is a necessity for cheap transportation.

Train Resistance.—The train resistance is decreased by the use of larger units.

Capacity.—Figures are given tending to show that the relative traffic capacity of narrow-gauge lines is not more than half that of medium- or broad-gauge.

Heavier Engines and Trains.—There is a general discussion showing the part played by the increase of train loads in reducing costs in the United States.

Cost of Operation.—The ton-mile costs on both Indian and Australian railways, as well as on those in the Argentine, are less on the medium- and broad-gauge than on the narrow-gauge, although the train-mile costs are often higher.

The lowest ton-mile costs on any railway in the Argentine are more than twice as high as they are in the United States. The saving in cost of operation, due to the use of larger cars and heavier trains, would be considerable, and would in itself go a long way toward meeting the interest charge on the additional cost of the wider gauge.

Cost of Maintenance.—Maintenance of way probably costs more per traffic unit on narrow- than on medium- or broad-gauge on any but lines of very light traffic.

Maintenance of equipment should be less on medium- or broad-gauge by reason of the larger capacity and, consequently, lesser number of locomotives and vehicles, provided there is business to warrant the use of these larger units.

Need of Efficient Transportation Machine.—The necessity of an efficient transportation machine to develop the country properly is pointed out, as well as the part played by the railways in developing the United States, the country most nearly comparable with Brazil and the Argentine.

There are two appendices: Appendix A gives a short account of the history of the gauge question in the United States, with some notes on its present status in India, Australia, South Africa, etc. Appendix

B contains a general description of the topography, physical characteristics, and business conditions of Southern South America, and the development and present situation of its railways.

It is shown that the development and growth of this area is most nearly comparable to that of the United States, and for its exploitation a transportation machine not less efficient than that of the railways of this country is necessary. The growth of the railways of the Argentine is reviewed in some detail, showing their development within the past few years from a series of little local lines which were not much affected by the gauge to a series of fairly extensive systems, where the distances to be covered and the necessary weights of trains to be handled are showing the deficiencies of the narrow-gauge. This growth, and the increasing distances from the seaboard to new areas being opened up, all point to the need of the most efficient transportation system for its proper development.

It is believed to be shown that the narrow-gauge is an inferior transportation machine; that, as far as new lines are concerned, there is little saving and no economy in their construction, and that this gauge is inadequate for the development of areas as large as those under consideration; that the 5 ft. 6-in. gauge offers little if any advantage over the 4 ft. 8½-in.; that, taken altogether, the latter, which has been adopted as the standard in North America, Europe, Western Asia, and Australia, is best adapted to the development of this region; that new lines should be built to this gauge, and that it is economically sound to make the expenditures necessary to change the existing narrow-gauge lines to this standard.

It is pointed out that the cost of changing the gauge of the existing lines cannot be considered entirely alone, and figures are presented showing the estimated additional cost of the whole system of standard-gauge lines at the end of 20 years. The figures tend to show that, including the cost of changing the existing narrow-gauge lines, and taking into account the additional cost of building the new lines of standard instead of narrow gauge, that the final standard-gauge system will have cost about \$5 000 per km. more than if the narrow gauge is perpetuated, and that, under certain assumptions, such as the greater need of stone ballast, the earlier requirements of double track, etc., on the narrow-gauge, this estimate would probably be materially reduced.

Figures are given, which, taking into consideration only those items which are readily estimated in money values, tend to show that the saving in cost of operation would be sufficient to cover the interest charges on the additional cost, and, besides all this, there is the fact that the narrow-gauge is an inferior transportation machine not adapted to the development of the transportation system of a continent.

The question of the gauge of railways was settled in North America many years ago, and, so far as regards this country, or even Europe, any discussion of the matter is purely academic. The development of railway transportation in the rest of the world, however, except in certain limited sections, is as yet in its infancy, the only sections of the vast area lying to the south of the United States where any substantial progress has been made being parts of the Argentine, Southern Brazil, Chile, Peru and Mexico, and these by lines of many different gauges, varying from 2 ft. 6 in. to 5 ft. 6 in.

The writer was asked recently to make a study and report on the gauge question, as affecting the future development of the Argentine, and, as the subject is a vital one in all countries south of the United States —or perhaps more correctly south of Mexico—it is thought that the presentation of those parts of the report which are of general interest may not be untimely. This, perhaps, is especially the case now, when the United States is beginning to be forced to look for foreign markets for its manufactures, with South America as the natural field for its enterprise, and when, consequently, the interest which American engineers are likely to have in this development may be of considerable importance.

The probable future growth of the southern part of South America may be fairly comparable with that of the United States during the past 50 years. The importance of the railways as a factor in this latter has been well stated by M. Colson, an eminent French engineer,* who points out how necessary cheap, efficient railway service is in the development of new countries. It is for this reason that American railway engineers, who have been intimately connected with the railway development of their own country, should be interested in the future of South America, as their own experience should enable them to be among the best judges of the necessities of countries which, like

* See p. 375.

those in southern South America, are entering on an era of expansion comparable only to that of their own.

Europe, like the United States, long ago adopted the 4 ft. 8½-in. gauge as the standard, but the gauge question is still being actively discussed in India, where the standard is 5 ft. 6 in., with a considerable mileage of 1 m. and less. Australia, where the gauges are 5 ft. 3 in., 4 ft. 8½ in., and 3 ft. 6 in., according to recent reports, has only recently decided to unify them and adopt 4 ft. 8½ in. as the standard; and Africa, unfortunately, seems to be definitely committed to what one eminent British engineer recently described as "the miserable 3 ft. 6 in. gauge."* China has adopted 4 ft. 8½ in., the Trans-Siberian Railway is 5 ft. 0 in., the South Manchurian Railway and the Korean railways are 4 ft. 8½ in., and the Japanese railways are mostly 3 ft. 6 in., with some 2 ft. 6 in., though it is stated that these latter are to be changed to 4 ft. 8½ in. as the result of their failure to meet the emergency caused by the Russian war.

It is, perhaps, true that the possible use of railways for war purposes can hardly be considered as an important economic factor in determining the basis on which they shall be designed—at least it is hardly as important in America, either North or South, as it might be in Europe or Asia—still the experience of the Japanese, as well as the English in India, tends to show the inferiority of the narrow gauge for transportation, especially in an emergency. In countries growing as rapidly as those of southern South America, this really means that it will not be long before the narrow-gauge lines will reach the same state of inadequacy to meet the natural development and growth of a new country as they have wherever they have been submitted to the emergency test of war.†

This is already apparent on some of the lines in the Argentine, and, in Brazil, the Paulista Railway Company is now reconstructing its narrow-gauge lines west of Rio Claro to 5 ft. 3 in., as it cannot handle the business offered, although the heaviest narrow-gauge equipment, both locomotives and rolling stock, is being used. Some further notes in regard to the experience of other countries with lines of various gauges are given in Appendix A.

* "The Railway Gauges of India," by Sir Frederick Robert Upton, *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV, p. 196.

† See, also, p. 341.

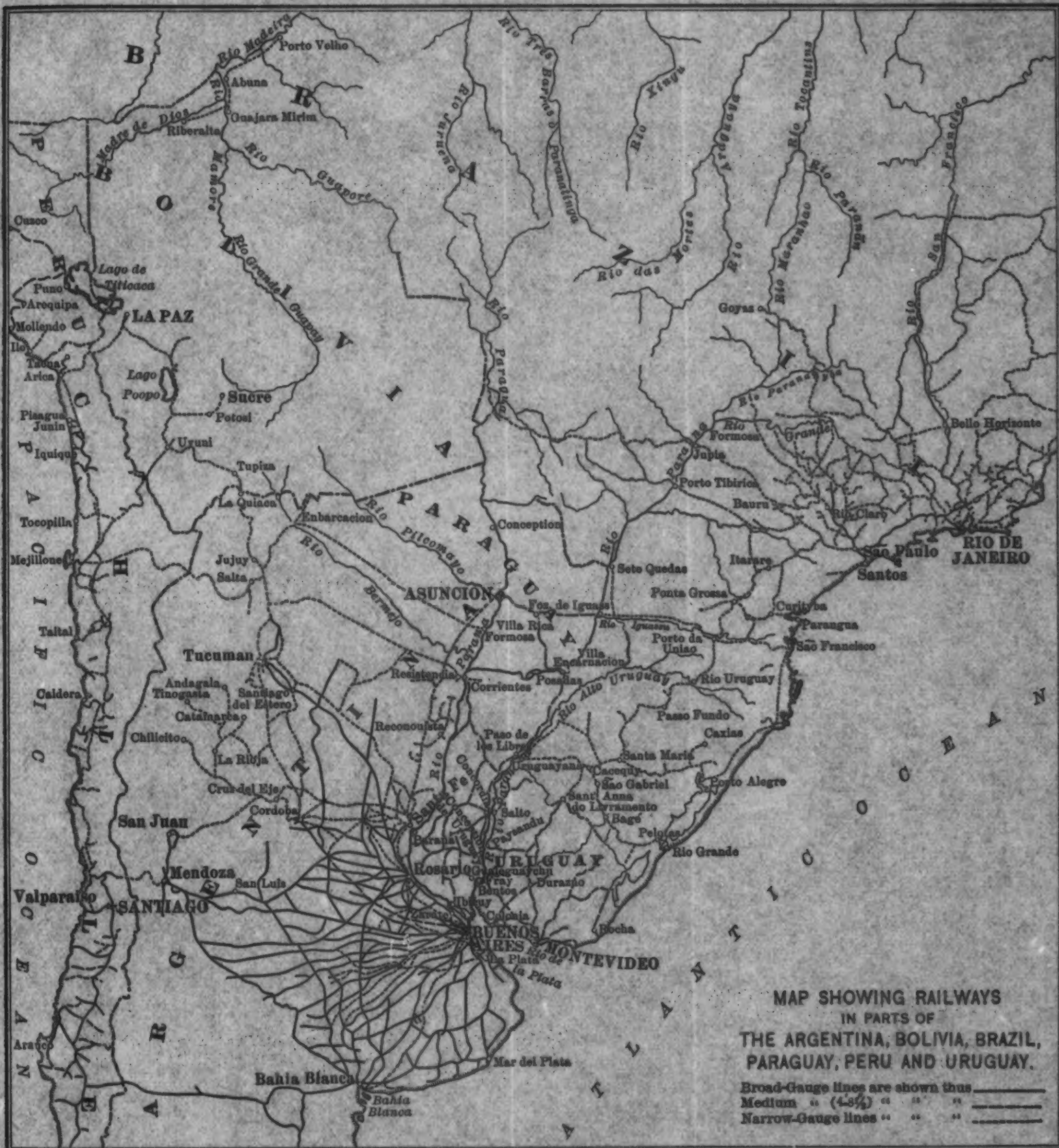
Some matters treated in this paper are, perhaps, not strictly technical, or may be considered somewhat elementary; but, as this subject is of interest to laymen as well as to engineers, and as the latter may often be called on to present the arguments, for or against the adoption of any particular gauge, to governments, the public, or investors, it has been thought desirable to include them all. In regard to some phases of the technical aspect of the subject, also, certain matters have been included which have long been accepted as almost axiomatic by engineers in North America, though it is the writer's experience that they are not always so accepted elsewhere.

The lengths of the lines of various gauges in the countries to the south of Mexico (which latter is practically all standard (4 ft. 8½-in.) gauge) are shown in Table 1.* Generally throughout the paper, when speaking of narrow gauge, it will be considered to include the 3-ft., 1-m. and 3 ft. 6-in. The medium or standard gauge is the United States, 4 ft. 8½-in. The broad is 5 ft. 3-in. and 5 ft. 6-in. It may be noted that the standard is 5 ft. 3-in. on the Irish Railways, and 5 ft. 6-in. on the Indian Railways.

TABLE 1.—Length, in Miles, of Railroads of Various Gauges in Countries South of Mexico.

	2 ft. 0-in.	2 ft. 6-in.	3 ft. 0-in.	Meter.	3 ft. 6-in.	4 ft. 8½-in.	5 ft. 0-in.	5 ft. 3-in.	5 ft. 6-in.
Guatemala	....	....	458	....	....	....	....	....	....
Salvador	....	....	100	....	....	....	....	....	....
Honduras	....	....	24	....	....	....	....	....	....
Nicaragua	....	....	....	....	172	....	....	....	....
Costa Rica	....	....	....	....	564	....	....	....	....
Panama	....	....	....	....	....	....	30	....	....
Colombia	....	....	340	63	18	....	....	....	....
Venezuela	110	....	24	....	199	....	....	....	....
British Guiana	....	....	....	....	....	60	....	....	....
Ecuador	....	....	....	....	278	....	....	....	....
Peru	....	....	262	8	15	994	....	....	....
Bolivia	....	468	54	....	....	....	....	....	....
Chile	....	554	....	2 359	279	517	....	....	497
Argentine	....	....	....	6 181	....	1 589	....	....	11 942
Paraguay	....	....	....	....	....	232	....	....	....
Uruguay	....	....	....	....	....	1 374	....	....	....
Brazil	....	....	....	11 315	....	....	....	286	....
Totals	110	1 022	1 257	19 926	1 525	4 766	30	286	12 439

* Compiled from "Universal Directory of Railway Officials."


MAP SHOWING RAILWAYS
IN PARTS OF
THE ARGENTINA, BOLIVIA, BRAZIL,
PARAGUAY, PERU AND URUGUAY.
Broad-Gauge lines are shown thus
Medium " (4-8½) " " "
Narrow-Gauge lines " " "
BRAZIL
BOLIVIA
PARAGUAY
ARGENTINA
URUGUAY
PERU
CHILE
PACIFIC OCEAN
ATLANTIC OCEAN
LA PAZ
ASUNCION
RIO DE JANEIRO
BUENOS AIRES
MONTEVIDEO
SANTIAGO
Sao Paulo
Santos
Valparaiso
Mendoza
San Juan
Tucuman
Rosario
Cordoba
Bahia Blanca
Mar del Plata
Porto Velho
Sucre
Potosi
Uyuni
Tupiza
La Quiaca
Jujuy
Salta
Corrientes
Posadas
Conception
Porto Alegre
Rio Grande
Curityba
Paranagua
Bello Horizonte
Goyas
Cusco
Puno
Arequipa
Mollendo
Arica
Iquique
Tocopilla
Taltal
Caldera
Arauco

Appendix A contains a table showing the length of the lines of the various gauges throughout the world, the total length of each gauge being, approximately:

Broad	145 300 kilometers	=	90 086	miles.
Medium	727 799 "	=	451 235	"
Narrow	183 416 "	=	113 719	"

The accompanying map, Plate IX, shows the railway lines of Southern Brazil, Uruguay, Paraguay, the Argentine, and parts of Chile, Peru, and Bolivia, and indicates the extent of each of the three gauges and their location. Speaking generally, it will be noted that the Brazilian railways are nearly all narrow-gauge, with the exception of parts of the important lines of the São Paulo and Paulista Railways, running into the interior from Santos, the line of the Central Railway of Brazil connecting São Paulo and Rio, and a line from this latter point into the interior, all of these being 5 ft. 3-in. The medium-gauge (4 ft. 8½-in.) lines occupy the strip of country between Brazil and the Parana River, including the whole of Uruguay and Paraguay, and the Argentine Provinces of Entre Rios and Corrientes. In the Argentine, the most intensively developed section—that lying between the City of Bahia Blanca on the south, and the Cities of Rosario, Cordoba, and Mendoza on the north—is occupied principally by the broad-gauge, the narrow-gauge railways occupying the section to the north of this, though in the zone between Buenos Aires, Mendoza, Cordoba, and Santa Fe, the territory is divided between the two, the broad-gauge having somewhat the advantage. Chile and Bolivia seem to be committed to the meter-gauge for all new lines, the former, however, has seven distinct gauges, and in Bolivia the Antofogasta Railway is 2 ft. 6-in. Peru, although fully as mountainous as any part of Chile or Bolivia, has practically adopted the 4 ft. 8½-in. gauge, its lines being among the highest in the world, with several hundred miles at elevations of more than 10 000 ft. above sea level, the famous Oroya Railway reaching 15 583 ft., the highest main line in the world. North of Peru the lines are nearly all narrow-gauge until Mexico is reached, the single exception being the Panama Railway, which is 5 ft.

A more detailed description of the railways of the Argentine, and

of the climatic, topographical, general physical conditions, and business of the southern part of South America is given in Appendix B.

Effect of Gauge on Location.—It is quite generally supposed that the narrow-gauge is better adapted to mountain location than the wide-gauge, because it permits the use of sharper curves and steeper gradients. Within the limits of practical railroad operation, this supposition is not warranted by the facts, when comparing meter and 4 ft. 8½-in., and there is very little difference with regard to 5 ft. 6-in.

Curvature.—Within the limits of curvature allowed on the meter-gauge roads of the Argentine, that is, with a minimum radius of 490 ft. (150 m.), it is quite possible to operate 5 ft. 6-in. gauge rolling stock, which, as a matter of fact, is handled every day in long trains with side-buffers in the yards at the terminals over No. 8 turnouts which have a radius of about 485 ft. (148 m.), and over main-line, No. 10 turnouts with a radius of 780 ft. (241 m.). There are said to be branches on some of the broad-gauge roads with curves of 490 ft. (150 m.) radius; and the line to the power-station at Lules, near Tucuman, over which broad-gauge trains are operated, has curves of less radius than this. The Southern has just located a line with curves of 656 ft. (200 m.) radius across the mountains beyond Neuquen.

The idea still exists in the minds of many people that trains can actually pass sharper curves on a narrow-gauge than on a broad-gauge. "Reduced to absurdity", this, of course, is so, but within the limits of practical railroad operation, on such lines as those under consideration, it is not, and there can be no question that trains of 5 ft. 6-in. gauge with center couplings can pass around any curves which it is practical to operate on meter-gauge lines built to do business. Sharp curvature limits the speed of trains, and in this respect it imposes quite as much, probably greater, limitation on narrow-gauge trains by reason of their lesser stability than it does on broad-gauge trains. The fallacy that sharper curves are not possible on broad-gauge lines probably arose in the early days, before the swiveling truck was introduced. The possible adaptability of the broader or medium gauge to the conformation of the ground has also quite generally been confused by the assumption that, because the gauge was broader the whole road and rolling stock must be of heavy construction; that is, with many people the term "light" railways has

been considered synonymous with narrow-gauge, whereas there is no reason that light rails and light rolling stock cannot be used as well on broad-gauge as on narrow. One has only to consider the rolling stock in use on British Railways 25 years ago, or even to a large extent in England to-day, with the little 5-, 8-, and 10-ton wagons, to realize this. The use of sharp curvature is by no means advocated as desirable, as the objections to its use are fully realized, but the objections to sharp curvature are no less on meter-gauge than on the medium or broad.

Properly designed rolling stock is, of course, a necessity in any case. Certain 40-ton shunting engines of the Cordoba Central Railway (meter-gauge) have difficulty in passing curves of 328 ft. (100 m.) radius in the yards, by reason of their stiff frames and long wheel base. The 850 000-lb. Mallets on the Atchison, Topeka and Santa Fé Railroad in the United States (4 ft. 8½-in.) are operated on curves of only slightly greater radius without trouble, and haul trains of standard Pullman equipment, the coaches being from 65 to 75 ft. long.

A locomotive of the Mikado (2-8-2) type, having a total weight of 285 000 lb. (220 000 lb. on drivers), built by the Baldwin Locomotive Works, has just been put in service on the railway of the Woodward Iron Company (4 ft. 8½-in.), on a line with 3% grades and 16° (110 m. radius) curves.

The New York City Elevated Railways (4 ft. 8½-in.) were built about 35 years ago, and are operated on an open steel viaduct over the streets of New York. Nearly a million passengers and about 1 000 trains a day pass over them. They have several curves of 90 to 125 ft. (30 to 40 m.) radius, and have an enviable record for safety, only one accident to a train in which the life of a passenger was lost having occurred in 35 years. These lines were formerly operated by steam, but now by electricity with multiple-unit trains.

On the lines of the New York City Subway (4 ft. 8½-in.), built about 5 years ago, an average of more than 1 000 000 passengers are carried daily. On these lines the radius of many curves is 147 ft., over which cars 51 ft. long and 9 ft. 0½ in. wide, are operated in 10-car trains by the multiple-unit system.

Table 2 contains some data for standard-gauge lines operating (many with a very heavy traffic) over sharp curves and heavy gradients,

some of them, as will be seen, rising to quite high elevations and thus indicating the adaptability of the 4 ft. 8½-in. gauge to mountain location.

TABLE 2.—DATA RELATING TO GRADIENTS, CURVATURE, ETC., ON CERTAIN RAILROADS.

	Grade.	Radius of Curves. In feet.	Radius of Curves. In meters.	Elevation of summit.	Remarks.
Canadian Pacific	4.49%	498	152	5 299 ft. (1740 m.)	
" New Line	2.20%	574	175		Spiral tunnels.
Atchison, Topeka and Santa Fé	3.30%	357	109	7 510 ft. (2 464 m.)	425-ton Mallet.
Denver and Rio Grande	3.80%	498	150	10 433 ft. (3 423 m.)	
Colorado Southern	3.50%	193	59		Georgetown loop.
Mersey Tunnel	3.00%				
Mexican Railway	4.75%				
Nitrate Railway of Chile	4.00%	298	91		
Central Railway of Peru				15 865 ft. (5 205 m.)	

On the Atchison, Topeka and Santa Fé Railroad—a transcontinental line of very heavy traffic—articulated locomotives of the Mallet type, weighing 425 tons (engine and tender), are in use.

Canadian Pacific: The old 4.49% grade has been operated by adhesion since 1885 up to within a year or two ago, when the spiral tunnels on curves of 574 ft. (175 m.) radius were built and the grade reduced to 2.2 per cent.

On the Nitrate Railway of Chile, 4 ft. 8½-in. gauge, a duplex articulated locomotive hauls 200 tons at 8 miles per hour on 3 to 4% grades (average 2.8%) with curves of 300 ft. radius.

In connection with the location of the line of the Wolgan Valley Railway in Australia, the whole question of gauge and the adaptability of standard (4 ft. 8½-in.) gauge to difficult mountain location was carefully studied by the Chief Engineer, Mr. H. Deane, who reported as follows:

"To bring the line within the region of practicable cost, a ruling grade of 1 in 25 was adopted, with curves of 5 chains [330 ft. = 100 m.] radius. There was no possibility of compensating for curvature, and the 1 in 25 grades occur, therefore, on 5-chain curves, so that the actual ruling grade may be said to be 1 in 22.5 not 1 in 25. A study of plan and section, as well as an inspection on the ground, will show how rigid were the conditions of the problem.

"Bound up with the whole question was that of gauge. Steep grades on a narrow gauge limit the load too much. It was anticipated that when the Company was in full swing, over 1 000 tons of goods would have to be conveyed over the line daily. It was clear, therefore, that the standard gauge must be adopted, especially as the Railway Commissioners had offered to lend their rolling stock if that gauge were not departed from. But how about curvature? it will be said. Was it not excessive? No. Not for the wagons, which were

daily hauled safely over the Camden Railway with its 5-chain curves. But what about locomotives? On the Western line, curves of 8 chains [528 ft. = 173 m.] radius were originally constructed, and had all been cut out because the wear of rails and flanges had been excessive.

"This question had to be solved by looking to the practice of other countries. In New South Wales the locomotives were too stiff. Some other type must be adopted.

"During an extensive journey around the world in 1894, I found numerous curves of 16 degrees, equal to 5½ [363 ft. = 111 m.] chains radius, one curve of 18°, equal to 4.8 chains radius and one of 22°, equal to 4 chains radius, on the Southern Pacific Railway system in the Western United States, and these were traversed by 8-wheeled coupled American locomotives of the Consolidation type. This was rendered possible by providing two of the pairs of wheels with broad, plain treads in place of flanging them. The curves mentioned have now been cut out, but they were worked for many years.

"In 1904 I travelled in a train on the main line of the Canadian Pacific Railway where one curve of 3½ chains [231 ft. = 70 m.] radius exists. All the Company's locomotives traverse this curve.

"On some of the mining branches of the Canadian Pacific Railway, where curves of 5 chains and grades of 4½%, equal to 1 in 22.5, exist, Shay locomotives are used.

"On the Tamaulipas Railway, a scenic line in California, there are curves of 70 and 80 ft. [21 to 25 m.] radius, the traffic being hauled by locomotives of the Shay type.

"On the Kandy Railway in Ceylon there are curves of 5 chains radius, the gauge being 5 ft. 6 in. These are negotiated by locomotives built by Kitson and Company, of Leeds. They are 6-wheeled, coupled, with bogie in front. The middle wheels have thin flanges, considerable play in the axle boxes is allowed, and the connecting rod and side rod pins are barrel shaped, so as to permit of the rods working out of the straight line."

As a further concrete example of the possibilities of operating on standard gauge over mountain lines with heavy gradients and sharp curvature, the following may be cited,* the line being part of one of the Transcontinental routes in the United States:

The main line of the Southern Pacific Railroad in California, between Bakersville and Mojave, a distance of about 68 miles (109 km.), is single track, with maximum grades of 2.2%, uncompensated against traffic both ways, and 10° 20′ (555 ft. = 169 m. radius) curves. There is one continuous curve with a total central angle of 566°, nearly all of which has a radius of less than 600 ft. (180 m.), 18 tunnels

* *Railway Age Gazette*, July 25th, 1913.

with a total length of 8 115 ft. (2 473 m.) and about 4 000 ft. (1 219 m.) of bridging. The summit is 4 025 ft. (1 227 m.) above sea level.

The engines used on this line are as follows:

Passenger service, Mogul (2-6-0) type:

Total weight of engine	166 320 lb.
" " on drivers	144 120 "
Tractive effort	33 320 "

Helper engines, Consolidated (2-8-0) type:

Total weight of engine	208 000 lb.
" " on drivers	187 000 "
Tractive effort	43 305 "

Freight engines, Mallet (2-8-8-2) type:

Total weight of engine	435 800 lb.
" " on drivers	401 000 "
Tractive effort	94 880 "

On a single day in January, 1913, 36 trains passed over this section, of which 16 were passenger trains handling 110 passenger cars, 6 305 tons, and 20 freight trains handling 886 cars, 30 725 tons, a total of 36 trains, 996 cars, 37 030 tons. So that, although sharp curvature is not by any means advocated as desirable, it can be shown that where the exigencies of the topography demand its use, standard-gauge equipment can be operated on it quite as well as on meter-gauge, and the possibilities of reducing operating costs by the use of larger engines far outweigh any small economies in the first cost of narrow-gauge by reason of slightly decreased width of roadbed and shorter ties.

Gradients.—As regards gradients, of course, the gauge makes no difference, the same power being required in each case to move the same weight and kind of train. Wider gauge, however, permits the design of larger locomotives, much beyond the limit of capacity of the narrow-gauge, and the wider firebox and freedom from the cramped space of the narrow-gauge is a decided advantage. It is a fact that standard-gauge locomotives are worked on short stretches, such as approaches to coaling stations, etc., by adhesion, on grades as steep as 10%, and Shay geared locomotives regularly on grades of 4 and 5%; and, as noted previously, there are several lines of heavy traffic

worked by adhesion locomotives on grades between 3 and 4%, and until only recently the Canadian Pacific had grades of 4½% on its main transcontinental line worked by ordinary locomotives. The writer knows of no steeper grade than this regularly worked by adhesion on narrow-gauge.

As in the case of curves, steep gradients are not advocated as by any means desirable, but the ability to operate trains on them is not affected by gauge, except in the one very important item against the narrow-gauge of the very limited capacity of the locomotives as compared with the wider gauge. The heaviest locomotive of the Mallet type, built for narrow-gauge, weighs 176 tons, as compared with 425 tons for the heaviest standard-gauge, or in the proportion of 1 to 2.4.

Speed.—At present there is little demand in the Argentine for speeds in excess of those which can comfortably be maintained on narrow-gauge lines in first-class condition, but the growth of the country is so great that it is rapidly reaching the point where fast speeds will have to be maintained over long distances in order to get passengers and mail to their destinations within a reasonable time. There is no reason to think that the Argentine will long remain content with inferior service, but will expect and will get as good service as is obtained in North America, with which region its transportation system will be most closely comparable.

A tabulation of the regular schedule of the principal long-distance and fast trains in the Argentine at the present time shows the maximum and average speeds to be as follows:

	Kilometers per hour: Maximum.	Average.	Miles per hour: Maximum.	Average.
Broad gauge..........	61.8	46.3	38.4	28.8
Medium "	30.0	28.4	18.6	17.6
Narrow "	39.6	29.0	24.6	18.0

It is well known that in Europe speeds of 50 to 56 miles per hour are maintained regularly for distances of 200 miles and more by a large number of trains daily. There are many fast trains in the United States, at least 25 or more daily, which do as well or nearly so, though they are much heavier.

On the transcontinental and other long-distance runs in the United States, varying from 1 000 to 2 500 miles, average speeds of 35 to 40

miles per hour are maintained, and daily trains between New York and Chicago on two roads average more than 50 miles per hour for the 1 000-mile run.

Most of the lines in the Argentine, and practically all those cited, have very light gradients and good alignment, whereas, in the United States, most of the lines, especially the transcontinental lines where they cross the Rocky Mountains, have sections of long heavy gradients with considerable curvature; so that, both actually and comparatively, the Argentine is far behind the United States and Europe in the matter of speed, and the narrow-gauge lines are far behind the broad-gauge. The medium-gauge lines of the Entre Rios and the North East Argentine Railways can hardly be considered for comparison at present, as they are not in good physical condition, but they already have in hand plans looking toward an improved service between Buenos Aires and the North which should enable them to run their trains through to Posadas or Corrientes in about 24 hours, or at an average speed of 30 miles per hour, or faster if necessary.

In a discussion before the Institution of Civil Engineers of Great Britain in regard to the gauge of Indian Railways, and in reply to a question as to whether high speed was necessary in that country, the statement was made that, "India, like every other country, had progressive ideas of speed, comfort, and safety, and these would have to be met." There can be no question, also, that this applies equally to the Argentine and Brazil.

For passenger service the narrow-gauge is hopelessly handicapped, and can never meet the requirements of modern railway service either in speed, comfort, or safety. An ordinary through passenger train in the Argentine will consist of about twelve coaches, part of which will be sleepers, diners, etc. Such a train in the United States would weigh between 700 and 800 tons loaded, but, owing to the generally lighter construction of the Argentine rolling stock, the total weight on the broad- or medium-gauge lines there would be about 500 tons, and on the narrow-gauge, with the same type of rolling stock, it would weigh about 400 tons, with less than three-quarters the capacity.

It is possible, on narrow-gauge lines, in first-class condition, with the best design of engine, to reach a maximum speed of 50 miles per hour on a level grade with a comparatively light train (250 to 300 tons).

On grades even as light as 0.5 to 0.6%, which are the ruling ones on most of the lines of heavy traffic of the Argentine, and are the grades on the Central Cordoba Railway line between Buenos Aires and Rosario, the best average speed would hardly be more than 40 miles per hour, which would mean that the Central Cordoba could not, at the best, make the journey in less than 4½ hours, as compared with the present actual time of the Central Argentine Railway of 5 hours, and the possibility of the latter doing it easily in 3½ hours, if necessary, that is, at an average speed of about 50 miles per hour for the run.

With a train of 400 tons, the sustained speed between stations on the narrow-gauge, with very good track, will not exceed 40 miles per hour, and the average speed of express trains, including stops not oftener than, say, once every 62 miles (100 km.) will not be more than 35 miles. This speed might be adequate, but it can only be attained on almost perfect track, and even then at a considerable sacrifice of the margin between safety and recklessness. The effect of every slight irregularity in the track is felt very much more on the narrow-gauge than on medium or broad, and in this respect the broad has an advantage over the medium in a country like the Argentine, where stone or gravel ballast is so expensive.

Stability.—The lesser stability of the narrow-gauge will be immediately realized when one stops to consider that the height of passenger coaches must be practically the same, no matter what the gauge is. Considering the gauge reduced to a minimum with the height remaining the same, it can be seen that a difference in level of the two rails of, say, ½-in.—less than that of many low joints in earth ballast—would be sufficient to overturn a sufficiently narrow and high vehicle, whereas it would have very little effect on a wide one, even at high speed. Then again, as soon as unevenness develops in the track, the shock caused by passing trains is greater on the narrow-gauge by reason of the high center of gravity, more height in proportion to width, and becomes worse much more quickly, thus increasing the cost of maintenance and making the hard-riding track so often noticed on narrow-gauge roads. Fig. 1 will, perhaps, help to emphasize this point, which is quite important and often lost sight of by those who compare the efficiency of rolling stock on one gauge or the other.

The growing realization of the fact that the speed and weight of modern trains are producing stresses, even in the very best of modern track, far greater than were formerly suspected, is shown by the action of the Canadian Society of Civil Engineers, which, recognizing the probable existence, due to increased loads, of stresses not satisfactorily distributed by the old short ties, has advocated a longer tie.

The Railroad Commissions of the States of Pennsylvania and New York have ordered a reduction in the speed of the fast 18-hour Chicago trains, due to the difficulty of keeping the track in sufficiently good condition to permit these high speeds. The following is from a recent editorial in a technical journal:*

"It seems to us worth while at this time to enter strong protest against any further increase in the loads which are imposed on the steel rails, track and bridges of American Railways."

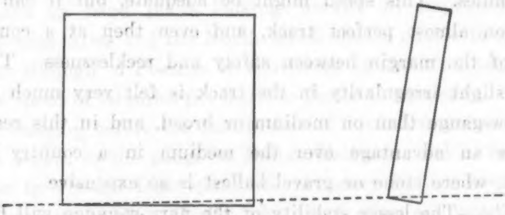


FIG. 1.

In the same journal† there is the following:

"The problem of weight is particularly difficult in regard to passenger locomotives [United States 4 ft. 8½-in. gauge] * * *. Already we have axle loads of 60 000 to 68 000 lb., and the impact effect or hammer-blow of such loads at high speeds is very severe upon joints, frogs, low spots, and the track in general."

The foregoing comments receive additional force when applied to narrow-gauge roads, where the much greater height in proportion to the width of the vehicles imposes additional shocks and much greater impact stresses than on the wider gauge.

It is well known, of course, that in the transmission of the power from the reciprocating parts of the locomotive engine to the wheels there is an unbalancing of forces which is only partly overcome by the counterbalancing of the wheels. Take, for instance, a Pacific type

* *Engineering News*, December 26th, 1912.
† July 11th, 1912.

engine with 60 000 lb. per axle, running at 60 miles per hour, the rail pressures under each wheel will vary from about 20 000 to 40 000 lb. at each revolution, according to the position of the counterbalance in relation to the rail.

An attempt to overcome this has been made in the class of engines known as balanced compounds, which have been used to a somewhat greater extent in Europe than in the United States. In this type of counterbalanced locomotive the four cylinders are arranged in a row, two inside and two outside the frame, and by the alternating action of the units of each pair they tend to overcome much of the unbalancing of forces noted above. It is hoped that the use of this type will tend to reduce the excessive track stresses with which we now have to contend, but, on account of the necessary arrangement of the cylinders, it cannot very well be adopted for the narrow-gauge, due to lack of requisite width and space for the arrangement of the cylinders.

As a concrete illustration of the maximum speed necessary in order to move goods traffic at even the rather slow average rate of 15 miles (24 km.) per hour, the following extract from a discussion of the problems affecting the transportation of fruit and vegetables in the United States by the Assistant General Manager of the St. Louis and San Francisco Railway, is of interest.*

Citing the run from New Orleans to Denver, 1 515 miles (2 438 km.), he states that at an average speed of 15 miles per hour, which seems quite slow for fruits and vegetables, the total time on this trip would be 101 hours. To get the actual running time there would have to be deducted:

Time lost at 10 district terminals, 1½ hours each	= 15 hours.
Time lost on each of 10 divisions of a single-track road for passing other trains, taking water and coal, say, 2 hours each	= 20 "
Total	35 hours.

This leaves 66 hours actual running time, or an average speed of about 24 miles (39 km.) per hour. In order to make this average speed, and considering the time lost for slow speeds on heavy grades,

* *Railway Age Gazette*, February 7th, 1913.

slowing down for orders (this particularly in the Argentine with the "Via Libre" system), signals, crossings, junctions, etc., makes it necessary to attain a speed of 40 miles (65 km.) per hour at least, on a good proportion of the distance.

To do that with the average freight train necessitates quite good track—it certainly would be verging on the dangerous to attempt it on even fairly good track on the narrow-gauge.

Rolling Stock.

One of the claims in favor of the narrow-gauge most frequently heard is that of the better utilization of the rolling stock; that the width of the cars is greater in proportion to the width of the gauge than it is on the wider gauges; that there is less dead load to paying load, and often that it is cheaper.

The first of these claims is undoubtedly correct, as far as it goes, but whether or not this is a real advantage is questionable, for, as already pointed out, the impact due to the oscillations caused by imperfections in the track (by reason of the proportionately greater height of the meter-gauge vehicles in relation to the width of gauge) is greater, and it is evident that this is also increased by the greater overhang. This causes a rougher-riding track and increased cost of maintenance, the narrow-gauge being forced to the limit in the matter of overhang, in order to provide reasonable capacity. That the other claims cannot be substantiated is believed to be shown by the following discussion.

Passenger Cars.—When it is considered that for passenger trains the capacity of the narrow-gauge is only three passengers in the same length of coach as will provide accommodation for four on either the medium or broad, it is seen at once that the slight extra cost of construction of the latter is more than justified. Comparing the medium and broad, the medium accommodates four passengers quite comfortably, two on each side of the aisle, the broad-gauge does the same, with some slight excess of room, and it is useless for passenger business to consider increasing the width of the broad-gauge, because, to be effective, this would involve three passengers on one side and two on the other, which, besides making unequal loading, would not be comfortable and, therefore, is out of the question. In the dining

cars the medium-gauge in the Argentine only allows for three passengers in the width of the car, though four can be accommodated by the broad, and this is the only place where the latter takes any advantage of its increased width. The latest type of all-steel diners in the United States, however, accommodates four passengers, two on each side of the aisle. In sleeping cars, whether of the Pullman type, or with compartments with transverse berths, as in general use in the Argentine, the medium-gauge is quite equal to the broad, and has generally less dead weight per passenger. When divided into compartments, the medium is wide enough to provide comfortable berths crossways of the car, and this is all the broad-gauge can do. If the Pullman type of car is used, the broad-gauge only gives more space in the aisle, which is of no advantage, so that the medium is at a slight advantage over the broad in the sleeping cars. On the narrow-gauge, compartments with berths across the cars are really impractical, although some have been built this way and the space quite ingeniously utilized, yet the berths are not long enough to be comfortable for the ordinary man. The narrow-gauge, therefore, is practically forced to the Pullman type of berths, placed lengthwise of the car, but utilizing only one side of it (as compartments have to be provided) so that they only have about one-half the capacity per unit of length of train that the medium-gauge has, and the dead load per passenger in narrow-gauge sleepers is about 2 tons, as compared with 1½ tons on medium or broad.

In order to determine, as far as possible, the relative capacity, cost, etc., of rolling stock on the different gauges in the Argentine, such information as could be obtained was collected, and is presented herewith for what it is worth. It is not as complete as might be wished, and, of course, comparison of rolling stock in any way is only valuable if the type of construction is the same. Practically, it is obvious, however, that given the same type of construction, the spacing of the wheels can make little if any difference, and these actual figures are given simply as confirmation to show that there is little difference in actual practice. The details of the passenger equipment are given in Table 3.

It will be noted that, in regard to cost, although the difference between the cost per ton as between narrow and medium is about 19%,

the difference in cost per passenger accommodated is 34%, both in favor of the medium-gauge.

Freight Cars.—The advocates of the narrow-gauge, though generally admitting that for passenger business the broader gauges have some advantage, almost invariably claim an equality in transportation of freight, within the limits of their capacity, and that their rolling stock is cheaper and has less dead weight in proportion to carrying capacity.

TABLE 3.—DETAILS OF PASSENGER EQUIPMENT.

	Narrow.			Medium.			Broad.		
	Weight, loaded, in tons.	Seats.	Cost, in gold.	Weight, loaded, in tons.	Seats.	Cost, in gold.	Weight, loaded, in tons.	Seats.	Cost, in gold.
First-Class	28	47	$12 250	38.3	68	$12 475	33.8	56	$11 800
Second-Class	27	67	9 900	40.5	102	9 700	32.2	64	8 590
Diners	61	48	29 000	37.7	40	17 000	33.6	32	13 050
Sleepers	27	12	11 760	37.0	20	15 520	32.2	23	12 770

COMPARISON OF UNITS.

	Narrow.	Medium.	Broad.
Cost per ton	$440	$356	$350
Cost per passenger	361	238	264
Weight per passenger, in tons	0.82	0.67	0.75

It is generally claimed that in the Argentine there is not much demand at present for either very long trains or cars of large capacity. Leaving this phase of the question for the present, it will be of interest to compare the freight cars of the three different gauges on somewhat the same basis as the passenger cars.

In regard to certain types, some additional data have been furnished to the writer by the Middletown Car Company, which has supplied a large number of narrow-gauge cars to the Government lines, and also to a standard-gauge line in Uruguay and to the broad-gauge port railway at Buenos Aires, these cars being of the same general types of construction. It is to be noted that the comparison of freight cars is somewhat difficult by reason of the greater variation in types of construction, the proportion of tare to total capacity

varying from 40% for well-designed cars to as much as 60% in some of the older types of less capacity. In the United States, also, the cars of large capacity show up to much better advantage than those of smaller capacity, as the underframes and draft rigging have to be the same for all, in order to allow all types of cars to be used in heavy trains. As far as possible, however, from the data of a large number of cars examined, the following are selected as representing good average practice in the Argentine for well-designed cars of fairly large capacity.

The question of what the result would be if the broad-gauge cars were changed so as to get the same proportionate overhang as medium-gauge, is discussed later, and the following comparisons are made on the basis of existing conditions, which it is unlikely will be changed.

It is to be noted that the greatest variation is to be found in the cost of the cars of the Rosario Puerto Belgrano Railway, which is considerably above the average. It is believed that, if sufficient information could be obtained as to the cost of freight cars on the other principal broad-gauge roads of the country, it would show little variation from the costs given of narrow and medium-gauge. The tons are metric, of 1 000 kg. (2 205 lb.).

TABLE 4.—DATA RELATIVE TO COVERED WAGONS (BOX CARS).

	Capacity, in tons.	Area of cross-section, in square feet.	Cubic feet.	Tare, in kilo-grammes.	Cost.
NARROW.					
Middletown Car Company......	30	51	1 695	12 200	....
Central Cordoba..............	25	45	1 340	10 500	$1 385
Central Northern.............	20	....	....	10 000	1.100
MEDIUM.					
Middletown Car Company......	30	70.7	2 545	15 380	....
Entre Rios...................	30	73.4	2 350	12 500	$1 470
BROAD.					
Middletown Car Company......	30	71.1	1 860	12 675	$1 400
Rosario Puerto Belgrano......	40	75.1	2 885	....	2 075
Southern.....................	40	....	2 220	15 120	1 875
Central Argentine............	40	....	2 185	16 350	1 875

It will be seen at once that, as far as regards weight of car to capacity in tons, there is little difference, with the exception of the Middletown Car Company's medium-gauge car, which has very large cubic capacity, the 30-ton cars averaging about 40 to 42 per cent. These small differences are less than are to be found between cars on any one line, of different types of construction.

It will be noted, however, that the narrow-gauge cars have generally only about two-thirds the cubic capacity of the wider gauges. The Middletown Car Company's broad-gauge car is quite a short one, being only 26 ft. long (the usual length being 36 ft., more or less), and, as a large proportion of the goods hauled in box cars is bulky rather than heavy, the extra cubic capacity is a most decided advantage. In the Argentine, as nearly as can be calculated from the statistics published by the Government, more than one-half the freight generally handled in box cars is of the former class, that is, classified by bulk rather than weight.

TABLE 5.—Data Relating to Stock or Cattle Cars.

	Capacity, in tons.	No. of animals.	Width, inside.	Floor area, in square feet.	Tare, in kilogrammes.	Cost.
Narrow.						
Middletown Car Company	30	....	7 ft. 11 in.	231	11 984	....
Central Northern	20	16	....	190	10 768	$1 240
Central Cordoba	22	20	8 ft. 0 in.	232	12 000	1 365
Medium.						
Middletown Car Company	26	....	8 ft. 5 in.	295	15 380	....
Entre Rios	22	20	7 ft. 10 in.	256	12 000	$1 475
Broad.						
Rosario Puerto Belgrano	40	24	7 ft. 3 in.	281	....	....
Pacific	28	....	9 ft. 6 in.	323	16 000	....
Pacific	28	....	9 ft. 6 in.	314	17 525	....
Average of several	40	....	8 ft. 6 in.	295	16 250	....

Stock cars are comparable on the basis of the relation between the floor area and the tare weight. The data in Table 5 are all the specific examples the writer has been able to get, and, in spite of being taken

from various sources, they show considerable agreement, the relation between floor area and tare weight for the three types being as follows:

Narrow .... 51.9 kg. per sq. ft. (114 lb.)
Medium .... 49.7 " " " " (109 ")
Broad .... 46.9 " " " " (103 ")

It may be noted that the Entre Rios car is very small for a stock car for 4 ft. 8½-in. gauge, and is not at all representative of the best practice, which would be at least equal to the others. The prices per ton of car and per square foot of floor area work out as follows:

Narrow: Central Northern. $115.15 per ton $6.75 per sq. ft.
" Central Cordoba.. 113.75 " " 6.40 " " "
Medium: Entre Rios..... 118.75 " " 5.75 " " "
Broad: Rosario Puerto Belgrano ....

TABLE 6.—Data Relating to Gondolas.

	Capacity, in tons.	Floor area, in square feet.	Cubic feet.	Tare, in kilogrammes.	Cost.	
Narrow.						
Middletown Car Company.......	30	250	804	10 900	....	....
Central Cordoba..............	25	244	....	11 000	$1 000	....
Middletown Car Company.......	30	250	....	12 210	1 345	All steel.
Middletown Car Company.......	20	....	....	9 070	840	....
Medium.						
Middletown Car Company.......	30	292	....	12 845	....	....
Entre Rios...................	30	284	....	11 250	$1 250	....
Rosario Puerto Belgrano......	37	308	700	14 400	....	....
Broad.						
Middletown Car Company.......	45	....	....	13 800	....	....
Central Argentine............	42	336	788	15 240	....	....
Western......................	45	340	1 630	15 200	....	....

For gondolas the comparison is best made between floor areas and tare weights. The cubic capacity is affected by the height of the sides, and as these may vary quite a little, this is not made, as all the details

are not available, also, the loads in these cars are often heaped up above the sides.

Narrow .... 45.8 kg. per sq. ft. of floor area (101 lb.)
Medium .... 43.5 " " " " " " " (96 ")
Broad .... 47.5 " " " " " " " (105 ")

TABLE 7.—DATA RELATING TO FLAT CARS.

	Capacity, in tons.	Floor area, in square feet.	Tare, in kilogrammes.	Cost.	
NARROW.					
Middletown Car Company......	30	262	9 300	....	....
Central Cordoba............	25	256	8 500	$825	....
Central Northern	20	217	8 145	....	....
Chile.........................	25	257	8 161	....	All steel.
MEDIUM.					
Middletown Car Company......	30	309	11 570	....	....
Entre Rios....................	30	258	11 000	$1 150	....
BROAD.					
Rosario Puerto Belgrano.......	40	359	....	$1 915	....
Average of seven cars.........	41	328	13 706	....	....

Narrow .... 34.0 kg. per sq. ft. of floor area (75 lb.)
Medium .... 40.0 " " " " " " " (88 ")
Broad .... 41.7 " " " " " " " (92 ")

It will be noted that, in comparing these four types of freight cars, so far as these figures show, there is very little difference between those of either gauge, what difference there is, however, being in favor of the wider gauges, especially for the box and cattle cars. The data are not as complete as might be wished, but are all that are available of Argentine rolling stock at this time, and, in view of the fact that the information has been taken just as it came, it is believed that it confirms what must be seen to be theoretically correct, that is, that gauge can make very little difference in cars of small capacity. A narrow-gauge box car with the wheels spread out can be used perfectly well on broad-gauge tracks which only differ from its own in being slightly wider. This extra width of trucks, as between narrow and

medium, only involves about 200 to 250 lb. more iron and steel, which at 6 cents per lb. is from $12 to $15, and this extra width is more than compensated by the increased stability and, consequently, lesser impact on the track.

The claim, therefore, that narrow-gauge cars are either cheaper or lighter in weight than cars of the same capacity and same type of construction for wider gauges, does not appear to be sustained by the facts, such variations as are shown being less than those between many cars of different make and design on the same railroad. It is of little value to include for comparison data relating to ordinary United States rolling stock, as this is all generally of heavy type, all the underframes and draft rigging being much heavier than those in use elsewhere, in order that they may withstand the strains due to the use of the cars in heavy trains, and, in addition, all cars are equipped with automatic power brakes, which are not in general use in the Argentine. The railways of the latter country, however, are beginning to realize the values of the saving due to the use of heavier cars and trains, and perhaps more so on the narrow-gauge lines. They are handicapped, however, by the fact that the underframes and draft rigging of all their old rolling stock, and even much of the new, are too weak to stand the strain. They are just beginning to be forced into the same position as the railroads of the United States were some time ago, that is, to adopt the same heavy type of underframe and draft rigging, no matter what the size of the car is, in order to be able to transmit the tractive forces without danger of breaking the train. When this becomes general practice, there will be a much greater discrepancy in the relations between the gross and paying loads for the lighter cars, and they will show up at a much greater disadvantage than they do even now.

Cost of Locomotives.—It is practically impossible to determine with mathematical accuracy the relative economy of locomotives for one gauge or another, as they vary so much in detail. Even supposing three machines as exactly alike as possible, except for the difference in the width apart of the wheels, which would give a difference in cost of only a trifling extra quantity of steel for the wider spacing, the performance would be so different as to vitiate any comparison. Even supposing three machines designed to be as nearly alike as possible,

but adapted to the gauge, there would still be so many differences that comparison would be questioned.

It seems to be a perfectly safe assertion, however, that, within the limits of the narrow-gauge, any reputable locomotive builder will undertake to build locomotives of each gauge having about the same power, capacity, and relative fuel economy for about the same price, irrespective of the gauge. There may be a little less steel in the narrow-gauge, but the labor for this slight difference is little, if any, less, and the greater ease of design for the broad-gauge more than compensates for this. There seems to be a vague idea in the minds of some advocates of the narrow-gauge that, by some means or other, narrow-gauge locomotives are more economical, whereas it is probable that the contrary is the case, as is noted in detail later. The hauling capacity of a locomotive, provided of course it is properly designed for power, is determined by the weight on the driving wheels, and a locomotive with 10 tons weight on its driving wheels will have exactly the same adhesion on a wide-gauge that it has on a narrow-gauge, so that, to pull the same gross load, the engines must have the same weight, if they are built on the same plan. A pound of coal, too, will not generate any more steam on the narrow-gauge than on the wide.

In order to get some idea at least of the comparative cost of locomotives for the three gauges in the Argentine, the writer obtained the figures given in Table 8, which, considering the different types of engines involved, different makers, etc., show a sufficient agreement in cost per ton at least not to disprove the argument that gauge makes little, if any, difference. The weights are per ton of engine and tender, light.

Size of Locomotives.—As at present worked, the 5 ft. 6-in. gauge has about the same capacity as the medium, though much heavier engines and trains are operated in the United States on the medium-gauge than elsewhere on any gauge. For reasons to be given later, it is believed that the capacity of the broad-gauge, measured by the size of engines and length of trains, will not be increased beyond that of the medium, unless by some entire change in transportation methods not even guessed at now.

The comparisons in Table 9, therefore, between the largest engines built to date (January, 1913), of the three types most generally in

use, will give a good idea of the tremendous advantage of the wider gauge in this respect for lines of heavy traffic.

TABLE 8.—DATA RELATIVE TO LOCOMOTIVES.

Description.	Class.	Cost.	Weight, in tons.	Cost, per ton.
METER GAUGE.				
Kitson, 1911	4-6-0	$12 500	50	$250
North British, Passenger, 1907	4-4-2	18 750	75	250
" " Cargo, 1907	4-6-2	19 500	78	250
Tank	0-6-4	11 500	47	245
Buenos Aires, Extension Passenger, 1906	4-6-2	18 000	78	230
" " " Cargo, 1906	4-8-0	17 250	73	235
Building Pacific Superheater	4-6-2	19 700	67	295
MEDIUM GAUGE.				
North British, Goods	2-8-0	$19 000	68	$280
" " Mixed Passenger	4-6-0	16 000	65	245
North American, Goods	2-8-0	17 000	56	300
" " Mixed	4-6-0	11 900	56	210
Shunting Engines	0-6-0	8 800	26	335
BROAD GAUGE.				
Tank	2-6-0	$9 360	39.5	$237
"	0-4-0	15 550	63.2	282
Passenger	4-4-0	21 400	70.1	302
Cargo	2-8-0	30 310	91.5	331

In the comparisons in Table 9 the 3 ft. 6-in. engines are given for the Pacific and Mallet types, as they are the largest narrow-gauge engines; the largest meter-gauge engines of these types thus far built, however, are quite a little smaller, the meter-gauge Pacific type being 19 by 26 in., with a total weight of engine and tender of 229 500 lb. and the Mallet 18½ by 29 by 22 in., with a total weight of 323 500 lb.

The freight engines of both types on the medium-gauge have practically double the capacity, and though the superiority of the passenger engine in tractive power is not so great, the grate area is just double, 70 sq. ft. as compared with 35 sq. ft., and the total heating surface, 3 936 sq. ft. as compared with 1 981 sq. ft., so that the capacity for sustained work, so necessary in passenger service, is very much greater in proportion. Size of boilers, heating surface, and grate areas are the limiting factors, of course, in all types of locomotives, and though the limit seems to have been nearly reached in these items in locomotives for narrow-gauge, it does not seem to be in those for medium-gauge. It

TABLE 9.—COMPARISON OF LOCOMOTIVES.

Type of Locomotives.	Gauge.	Weight of engine and tender, in pounds.	Weight per driving axle, in pounds.	Diameter of driving wheels, in inches.	Cylinders, in inches.	Maximum tractive power, in pounds.	Maximum load, 0.6% grade, in tons.	Remarks.
Passenger, Pacific, 4-6-2 type	4ft. 8½ in.	469 190	57 440	74	24 by 32	42 000	2 880	Standard-gauge: Baltimore & Ohio R. R. *Engineering News*, July 11th, 1912, p. 49.
	Meter.	259 800	35 330	62	21 by 28	28 800	1 564	3 ft. 6-in. gauge, South African Railways, Am. Loc. Co. Catalogue, p. 35.
Freight or mixed, Mikado, 2-8-2 type for fast or heavy service	4ft. 8½ in.	539 480	58 300	63	28 by 32	27 460	3 196	Standard gauge: Erie R. R., 1912. *Engineering News*. July 11th, 1912, p. 51.
	Meter.	246 600	27 625	49	20 by 26	28 900	1 564	Meter-gauge: Am. Loc. Co. Catalogue, p. 23.
Freight, articulated (Mallet), for heavy, slow service	4ft. 8½ in.	850 000	55 000	57	28 by 36 by 32	111 600	6 324	Standard-gauge: Atchison, Topeka & Santa Fe R. R. *Engineering News*, May 4th, 1911, p. 548.
	Meter.	352 000	32 000	46	18 by 25½ by 26	57 700	3 266	3 ft. 6-in. gauge: South African Railways. Am. Loc. Co. Catalogue, p. 77.

is stated that there is under construction now a medium-gauge locomotive of 160 000 lb. tractive power, nearly 50% more than that of the large Mallet given in Table 9.

Relative Capacity of Wide- and Narrow-Gauge Lines.—The comparisons between the capacities of the meter- and broad-gauge lines of India given in Table 10 were made by Sir Robert Upcott,* the figures covering sixteen of the principal railway lines.

TABLE 10.—CARRYING CAPACITIES, COMPILED FROM RAILWAY ADMINISTRATION REPORT, COVERING SIXTEEN LINES IN INDIA: EIGHT OF 5 FT. 6-IN. GAUGE AND EIGHT OF METER-GAUGE.

		Ratio.		
		Standard-gauge	to	meter-gauge.
Vehicle capacity only	Passenger	1.5	to	1
	Goods.	1.6	to	1
Same, taking speed into account	Passenger	1.9	to	1
	Goods.	1.9	to	1
Gross weight of trains	Passenger	1.4	to	1
	Goods.	2.0	to	1
Same, taking speed into account	Passenger	1.9	to	1
	Goods.	2.4	to	1
Vehicle mileage, loaded and empty	Passenger	1.9	to	1
	Goods.	2.8	to	1
Same, taking speed into account	Passenger	2.4	to	1
	Goods.	3.3	to	1
On actual number of passengers carried 1 mile		2.0	to	1
On actual number of tons carried 1 mile		4.7	to	1

Engines on broad-gauge, average 7 838 000 ton-miles per annum.
Engines on narrow-gauge, average 3 962 000 ton-miles per annum.

The following extract gives the opinion of an important official of the inferiority of the narrow-gauge lines in India, where the problem is very much the same as it is in the Argentine, namely, the movement of very large quantities of grain in a short space of time.

"It has been clearly brought out, writes Major-General Kennedy, by the facts of the present season's famine traffic, that occasions such as the present may arise when, in order to feed a large population, grain has to be carried over single lines of railway for long distances, in quantity sufficient to strain to the utmost the resources of the best appointed lines of the standard 5½-ft. gauge and, as it may be assumed that narrow gauge lines would be far less efficient, the unsuitability of the metre gauge for long lines of through communication cannot be put out of sight when thought is taken for the proper and efficient administration of a country liable to be affected with drought and famine.

* "The Railway Gauges of India," *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV, p. 202.

"Although the railway lines concerned in the present case, *i. e.*, the East Indian, the Great Indian Peninsula and the Madras Railways, have failed to do all that was required of them, or that was needful to meet the full necessities of the case, they have, nevertheless, it may be truly said, largely contributed to save Southern India from a very great calamity, but they have not done this with any margin to spare, or without some injury to particular sections of general trade or to individual traders. The standard gauge lines have been barely able to satisfy essential and urgent demands, and under similar circumstances narrow gauge lines would probably have broken down altogether. Indeed, it is known that in the Southern part of the Madras Presidency considerable inconvenience arose at a break of gauge.

"We may, with the memorial of the Bombay Chamber of Commerce before us, take it as proved that narrow gauge lines are unable to meet the sudden demands likely to be made upon them in India, and that they are far more expensive to work than broad gauge lines. At first sight it is difficult to see how the latter statement can be true, but we think we can clear up the point in a moment. When a given quantity of goods, say grain, has to be conveyed over a railway, two systems of conveyance may be adopted, that is to say, we may run large trains at long intervals or small trains more frequently. Now, on the broad gauge lines in India a goods train complete weighs on the average 505 tons, but on the Rajpootana State metre gauge railway the average goods train weighs but 200 tons, so that the broad gauge trains are really 2½ times as heavy as the narrow gauge trains, while their speed is higher."

Economy of Heavy Engines.—The following statement, recently made by J. J. Hill, F. Am. Soc. C. E.,* one of the foremost railroad executives in the United States, shows the value and importance of heavy engines and wagons of large capacity for the development of cheap and efficient transportation:

"Heavier rails, larger engines, cars of greater capacity, increased train movement and the full utilization of equipment have kept business moving. The density of traffic in England, France and Germany should be as much greater than in the United States as the density in the middle exceeds that in the far western states. Yet here are the facts:

"Ton Miles per Mile of Road.

France	496 939
United Kingdom	529 622
Germany	827 400
United States (1910)	1 071 096

* *Railway Age Gazette*, December 20th, 1912.

"* * * Our railroads move 272 ton miles of freight per dollar of net revenue, where the United Kingdom shows only 58, Germany 172 and France 88.

* * * * * * * * *

"Transportation costs the public from one-third to one-half as much here as in Europe. This cheapness is not purchased at the cost of the workingman. In 1910 the average daily earnings of railway employees in the United States were more than twice as great as in the United Kingdom, and two and three-quarter times as much as on the Prussian-Hesse system in Germany."

There has been some criticism of the exact figures used by Mr. Hill, and of the use of certain units as indices of efficiency, but, making every allowance for this and the fact that such criticisms of details are always possible, in view of the different conditions and manner of computing and compiling statistics, there is not enough difference to affect the main argument, which is, that the use of heavy rolling stock has been the principal cause of the very much lower costs of transportation in the United States than elsewhere.

Fig. 2,* relating to the improvement in operating efficiency of the Chicago, Burlington and Quincy Railroad during the past 10 years, shows with great clearness the possibilities of reduction in operating expenses by the increase of train loads. During this period, as shown by the diagram, the business of the road was nearly doubled (98.3% increase), though there was at the same time an actual decrease of 9.8% in the number of trains required to handle it. Part of this good showing was made by increasing the car loading, that is, the revenue-tons per car-mile, by 45.5% and the remainder was due to increase in the capacity of the locomotives, reduction of grades, and the maintenance of the roadbed and equipment in a high state of efficiency. The average capacity of cars was increased from 23 to 38 tons. The heaviest type of locomotive in use in 1901 developed about 20 000 lb. tractive power; the Mikado (2-8-2) type now in general use has a tractive power of 60 000 lb. The average train load is now 438 revenue-tons per train-mile, as compared with 200 tons in 1901.

Although Canada is somewhat behind the United States in the matter of economical handling of freight, it is far ahead of any other country in this respect. In New South Wales, where they are vig-

* *Railway Age Gazette*, January 18th, 1918.

orously agitating the double-tracking of many of their lines, the traffic density is only 226 906 ton-miles per mile as compared with 731 776 for Canada, the average train loads being 90 tons and 325 tons, respectively, so that actually more trains are being run in New South Wales to handle a smaller volume of traffic than in Canada, the freight-train-miles per mile of line in New South Wales being 2 512 as compared with 2 252 in Canada, in spite of the fact that the density on the latter is 220% more than on the former, and the average freight charge in Canada is 0.75 cent as compared with 1.78 cents in New South Wales.

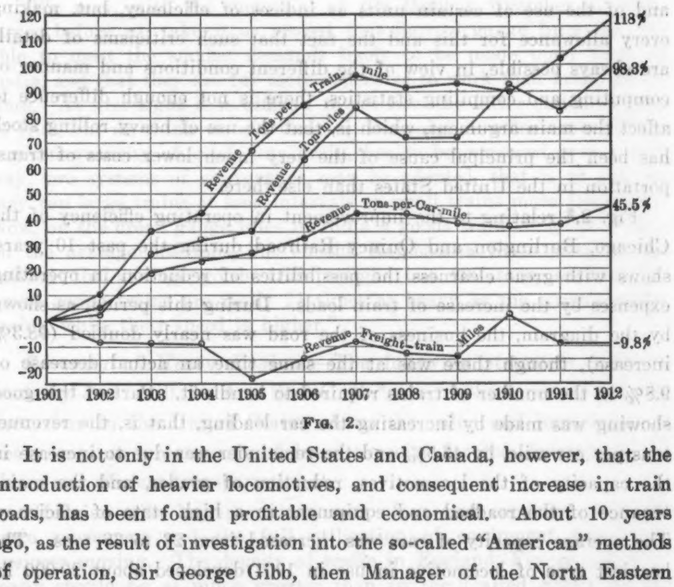


Fig. 2.

It is not only in the United States and Canada, however, that the introduction of heavier locomotives, and consequent increase in train loads, has been found profitable and economical. About 10 years ago, as the result of investigation into the so-called "American" methods of operation, Sir George Gibb, then Manager of the North Eastern Railway of England, introduced changes on that line which have resulted in the increase of the average freight train load from 60 to 95 tons, and on mineral trains from 114 to 184 tons. The result of this* is seen in the fact that for the last half of 1912 a dividend was announced at the rate of 7½% per annum, more than twice the average for British railways.

* *Railway Age Gazette*, March 28th, 1913.

In Germany, also, according to a statement of Mr. Hammer,* there is a growing realization of the necessity for the use of larger locomotives. On the Prussian State Railways (as elsewhere), owing to the continued tendency to increase the dead weight of passenger equipment, both actually and per passenger, in order to provide the better accommodations demanded by the traveling public and also the greater strength required for increasingly high speeds, the average weight per seat on passenger equipment has increased from 0.27 to 0.37 tons, or 37%, in the last 20 years. The introduction of heavier locomotives has reduced the train and locomotive mileage by eliminating many assistant engines, and also the axle mileage by the introduction of heavier wagons. The coal consumption was reduced from 60.05 tons in 1907 to 53.50 tons in 1910 per 1 000 000 ton-kilometers, in spite of the fact that the average speed was considerably increased. The ton-mileage of empties was considerably reduced during this period, and the tariffs were also reduced, yet notwithstanding this the consumption of coal for each 1 000 marks of receipts was reduced from 5.14 tons in 1907 to 4.65 tons in 1910. Mr. Hammer attributes these and other favorable results almost entirely to the use of larger locomotives and consequent increase of train loads.

In some instances in the United States, as pointed out later, the tendency to increase train loads has been carried beyond the economic limit, but these excesses (involving train loads as great as 8 000 tons) do not vitiate the general argument, that there must be a considerable increase in the Argentine to meet the necessities of modern transportation and to carry bulk freight long distances at rates which will enable the hitherto undeveloped sections in the interior of South America to enter the markets of the world, and that the necessary increase is not possible on the narrow-gauge.

Saving in Fuel.—The saving in coal consumption, made possible by the use of some of the more recent types of heavy engines, is also shown by the following quotation from a recent paper† by Mr. O. S. Beyer, Jr. In commenting on fuel, he said:

"Numerous tests and service records have revealed that large superheater Mikado locomotives which have been placed in service recently haul trains of 45 and 50% greater tonnage with the same amount of

* *Bulletin*, International Railway Congress, March, 1913.

† Am. Inst. of Mech. Engrs., Vol. 34, 1912, p. 1301.

coal that was formerly consumed by the Consolidation locomotives they replaced. Even the coal consumption of Mallet engines with grate areas up to 100 sq. ft. has not grown in any way proportionate to the increase in their hauling capacity. Modern engines when running at shortened cut-offs over those portions of the road other than the ruling grades exhibited a still greater economy than when working on the heaviest grades. Some service tests of recently built Mikado engines on the Delaware, Lackawanna and Western Railroad clearly demonstrated these facts. Their economy in fuel consumption as compared with that of the old Consolidation type, both operating over heavy grades at full load, being 20 per cent. The economy effected over easy grades while running at shortened cut-offs was 39.3 per cent., almost twice as much. The average was 29.1 per cent."

Weight of Trains.—Table 11, showing the records of heavy trains actually hauled in the ordinary course of business in the United States, will give some idea of the present state of the art there. The records are all within the past 3 years, and the length of the run varies from about 125 miles on the low gradients of the Virginian and Pennsylvania Railways to 50 to 65 miles on the other lines with steeper gradients.

TABLE 11.

Railroad.	Type of locomotive.	Train behind tender, in tons.	Gradient. Percentage.
Denver and Rio Grande	Mallet	500	4.00
Southern Pacific	"	1 250	2.20
Canadian Transcontinental	"	1 212	2.20
" "	"	4 290	0.40
Pennsylvania Railroad		8 778	0.23
" "	Consolidation	7 644	0.20
Virginian Railroad		6 023	0.20
" "	Mikado	7 562	0.20
Lake Shore and Michigan Southern		7 433	

The last five trains carry coal or iron ore in all-steel gondolas, all of the same type and of 100 000 lb. capacity, having 85 to 100 cars in a train.

Wider Rolling Stock for Broad-Gauge.—This brings us to the consideration of possible benefits to be derived from the broad-gauge, if advantage should be taken of the extra width of track to get the same proportionate width of cars as the medium- or narrow-gauge.

Experience has shown that the width now in use on the meter-gauge, combined with the height of passenger coaches, box cars, and cattle cars, is practically at or even beyond the limit. One has only to travel

on meter-gauge lines, even where the track is well kept up, to realize that as soon as any attempt at speed is made the oscillations are quite violent, and if the track is out of line and surface, speed becomes positively dangerous.

On the standard-gauge, which is well named the normal in many countries, the happy medium appears to have been most nearly attained by proper adjustment of the size of the vehicles to the width of the track. The clearance diagrams, Fig. 3, which are those established for the Argentine, show the ratios in Table 12 between the clearances on the three gauges; and it may be noted that the clearances for medium-gauge are ample for the largest equipment in use in the United States.

TABLE 12.

Gauge.	Permissible width. Argentine.	Width of track.	Ratio.
Broad	11 ft. 1¾ in.	5 ft. 6 in.	2.0 to 1
Medium	10 ft. 10 in.	4 ft. 8½ in.	2.3 to 1
Narrow	10 ft. 6 in.	3 ft. 3⅜ in.	3.2 to 1

TABLE 13.—RATIOS BETWEEN THE ACTUAL WIDTHS OF VEHICLES AND WIDTHS OF TRACK.

Gauge.	Actual width of vehicle.	Width of track.	Ratio.
Broad	10 ft. 6 in.	5 ft. 6 in.	1.91 to 1
Medium	10 ft. 2 in.	4 ft. 8½ in.	2.16 to 1 (Freight cars only 9 ft. 4 in.)
Narrow	9 ft. 1 in.	3 ft. 3⅜ in.	2.77 to 1

The statement is often made—and at first sight apparently with a certain amount of reason—that the broad-gauge has not taken proper advantage of the additional width of gauge. It has already been shown that there would be no advantage in increasing the width of passenger coaches. Up to the present there has been little demand for freight cars of greater capacity than 100 000 lb. (45 000 kg.), but, if there were, it could be met, up to at least 20 to 30% more, without increasing the width of any type. The only possible advantage of the increased width would be that, theoretically, a reduction in cost would be possible, because the nearer the body of the car approches a square in plan the more economical it is to build, and in this the 5 ft. 6-in. gauge has a

superiority over the meter-gauge which more than compensates for the cost of the additional width of track. Between the present width of the broad-gauge rolling stock (10 ft. 6 in.), however, and the width to make it proportionately equal to the medium-gauge, say 12 ft., there

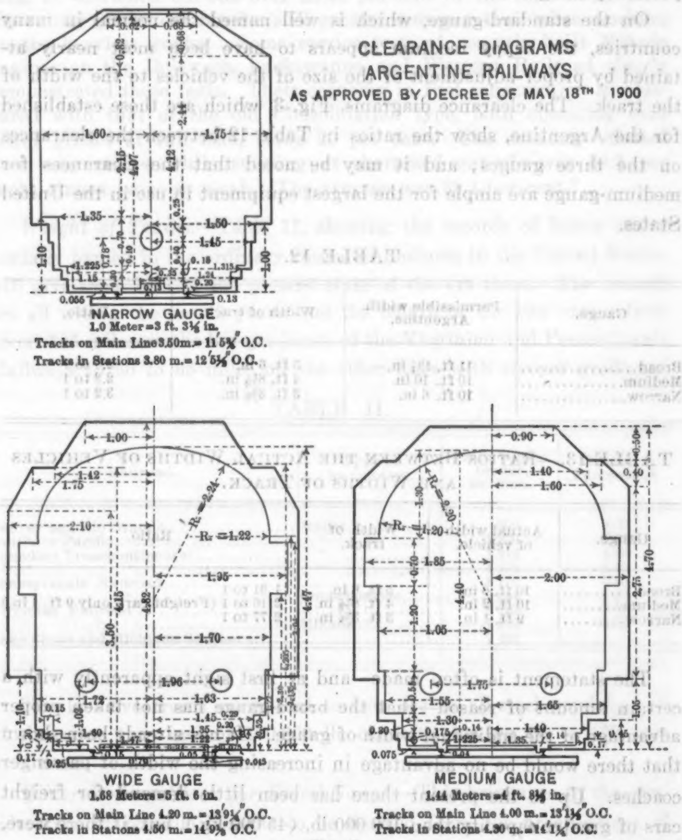


FIG. 3.

would be little saving in the cost of the vehicles, and no advantage in carrying capacity, even though the increased cost of additional clearance required for overhead structures, tunnels, etc., is practically negligible in the Argentine, as there are so few. There has recently been

designed for the Norfolk and Western Railway, and placed in service, high-sided steel gondolas of 100 tons capacity, weighing light about 65 000 lb., and having a simplified form of six-wheel truck. These cars are 10 ft. 4½ in. wide, over all, and their use indicates the possibility of a long step in advance in the operating capacity of the standard (4 ft. 8½-in.) gauge.

It has been suggested, however, that much larger locomotives would be possible if the wider clearance were allowed, and that, in view of the continuous and insistent demand for larger engines and more power to haul faster and heavier trains, this would be a most decided benefit. The average total weight on drivers, of the locomotives of the United States, increased from about 69 000 lb. in 1885 to more than 180 000 lb. in 1907, and reached a maximum of 316 000 lb. in that year. (The latest Mallet, 1912, has 550 000 lb. on drivers.) The average axle loads increased from 22 000 to 48 000 lb. in the same period, and since then have increased still more, being now, in at least one case, nearly 70 000 lb., and this on passenger engines running at high speeds (Pennsylvania Railroad, 4-4-2 with 68 000 lb. per driving axle). In view, therefore, of the general demand throughout the world for bigger types of everything, and most particularly in the transportation line, it is not surprising that the average person sees no reason for not continuing to build larger and more powerful locomotives, and that, in view of the fact that in some respects we seem to have reached the limit on the medium-gauge, the 5 ft. 6-in. looks attractive, and especially so in the additional width which might be utilized for the power plant. Reference has already been made to the statement that there is now under construction a locomotive of the Mallet type which is to have a tractive power of 160 000 lb., as compared with about 110 000 lb. for the largest now in use, thus marking a long step forward in the ability of the 4 ft. 8½-in. gauge to handle the heaviest traffic. There are some signs, however, that in some cases in the United States the length and weight of freight trains have reached or even passed the economic limit, and that there may be a reaction from the extremely heavy loadings, though by no means to reduce them to anything like the low standards of Europe or South America.

It has been pointed out* that these extremely large engines have

* *Engineering Record*, February 20th, 1909.

greatly increased the cost of bridges, and that heavy locomotives, because of the increased stresses in the track, increase the cost of maintenance. High speed, combined with heavy weights, is the cause of far more rail failures than poor steel. Lighter locomotives, shorter trains, and higher speed will give more service from the cars and more satisfaction to the shipper, who is not altogether in favor of holding trains for the last car the engine can haul.

In an editorial in a technical journal* relating to the controversy over the gauge question in Australia, the question of the advantage or otherwise of a broader gauge in the United States was examined; not, of course, with any idea of changing, but purely for the purpose of discussing what advantages there might have been had a wider gauge been adopted from the beginning, and the conclusion was that there would have been no benefit. Considering the matter further† the same journal gives the following reasons for thinking we have reached the economic limitations of the size of the locomotives and weights of trains:

Long heavy trains are only desirable if they save money.

The big locomotive and long train have been the principal factors which have enabled American railways to pay double the wages paid in Europe and at the same time reduce the ton-mile cost of hauling freight far below that of any country in Europe.

The principal saving in operating expenses by reason of heavy, long trains is in saving in wages of train crews, which item amounts to about 12½% of the total operating cost.

Legislation and labor organizations are nullifying this saving by making the pay more or less proportionate to the weight of the train or the size of the engine.

An argument in favor of long trains has been that the cost of controlling a train is practically the same whether it be long or short, but train despatchers as well as train crews have much more trouble getting very long trains over the road in time.

Excessive length of trains may and does interfere with their prompt and efficient handling at terminals or passing sidings, causing delay to other trains.

* *Engineering News*, December 7th, 1911.

† *Engineering News*, July 18th, 1912.

Heavy locomotives and high speed increase cost of maintenance very rapidly as they pass the normal; 125-lb. rails of open-hearth steel, with special alloys, are already being considered to meet the demands of heavy traffic, and this expense is a charge against the heavy locomotive.

Probably the most important consideration of all, however, is the item of rolling stock repairs, which has increased very rapidly with the increased weight of train and power of the locomotive. A hauling power of 25 tons and upward cannot be transmitted through a long freight train without exerting enormous stresses upon draft gear, couplers, car sills, floors, etc. It is not the mere push or pull of the locomotive, but the transmission of the power in a series of waves or shocks, especially on lines with broken profiles.

This matter of the proper design of rolling stock, and the strengthening of the underframes and draft rigging, is one which is receiving considerable attention, but, of course, the raising of the general standard of efficiency in a matter of this kind is very gradual, and in the meantime there is a temporary halt.

Enough has been said, however, to show that the 5 ft. 6-in. gauge offers no advantage in allowing the use of heavier cars or trains than are now in general use or in contemplation on the 4 ft. 8½-in., and the latter is capable of much further expansion along these lines, even beyond what are to-day considered to be the economic limits.

Train Resistance.—The term "train resistance" is used to denote the combination of forces which have to be overcome to produce the movement of the train, these forces being affected by the character of the road and vehicles and, therefore, to some extent, by the gauge.

Train resistance is of two kinds: that due to the internal resistance of the train, and that due to gradients and curvature. Numerous experiments have been made to determine the amount of the former, with varying results due to the complex nature of the problem, but one fact is firmly established, that the resistance per ton of train is much less for loaded cars than for empties, and less for heavier cars than for light ones. For a train of loaded 50-ton coal or mineral cars, all of the same type, the resistance, at ordinary freight-train speeds on straight, level track, is often as low as 3 lb. per ton, whereas trains of miscellaneous cars in the United States, including a fair proportion of cars as light as 30 tons and some empties, will have a resistance as

high as from 7 to 10 lb. per ton, 7 lb., however, being about the maximum with fairly good conditions. There is no doubt, therefore, that in this respect the narrow-gauge, with its much lesser average weight per axle, is at a disadvantage when compared with the broad-gauge, but just how much it is impossible to say.

The following table shows the effect of the capacity of cars on the loads which a consolidation engine (118 tons) can haul on a 0.5% grade at 20 miles per hour. The capacity includes weight of car and load, and the tonnage is based on the formula of the American Railway Engineering Association ($R = 2.22\ T + 121.6\ C$):

20-ton cars..................	1 570 tons.
30 " ".....................	1 775 "
40 " ".....................	1 910 "
50 " ".....................	2 010 "
60 " ".....................	2 090 "
72 " ".....................	2 160 "

The resistance due to grade is the force required to lift a certain weight a certain height; its computation is an exact mathematical proposition, and is not influenced in any way by gauge.

The resistance due to curvature is probably slightly less on narrow-gauge than on broad-gauge. It is of two kinds: one is the friction of the flanges of the wheel on the rails, which is probably in direct proportion to the force required to change the direction of a body tending to move in a straight line, this being a function of the speed and weight, and having nothing to do with the gauge; the other is the slipping of the treads of the wheels on the top of the rails, and this varies with the width of the gauge. That the latter is a comparatively unimportant factor is realized when one considers that, for example, on a curve of 400 ft. radius with a central angle of 90°, the difference in the length between the outer and inner rails for 4 ft. 8½-in. gauge is 7.39 ft. and for the meter-gauge, 5.15 ft. On a curve of 1 000 ft. radius with a central angle of 30°, which is about an average curve in the Argentine, the difference between the outer and inner rails would be:

Broad	2.88 ft.
Medium	2.47 "
Narrow	1.72 "

When it is considered that about 90.4% of the alignment in the Argentine, including in this all the mountain lines, is on tangent, this additional resistance due to the narrower gauge is practically negligible. On a strictly mountain line, with very heavy and continuous curvature, this additional resistance might be considered perhaps to be equal, roughly, to the additional resistance of the smaller class of rolling stock, but, for the average line, it would be considerably less. On the other hand, locomotives for the narrow-gauge, of equal weight and power with those of wider gauge, must necessarily be longer, and, therefore, the resistance encountered by them in passing around curves would be somewhat greater than with the shorter engines for wider gauge.

Taken altogether, therefore, it seems quite safe to say that the resistance per ton of train on narrow-gauge lines is greater than on wider gauge, and the cost of operation is thereby increased, though it is hardly possible to calculate the amount.

Costs of Construction.

The reasons which are most generally advanced by the advocates of narrow-gauge for the supposed economy in first cost of construction are as follows:

1.—The use of sharper curvature is supposed to permit better adaptation of the line to the conformation of the ground.

2.—The cost of earthwork is supposed to be less:

  a.—For reason No. 1.

  b.—Because they are narrower.

3.—The cost of bridging is supposed to be less:

  a.—Because they are to carry lighter loads.

  b.—Because they are narrower, especially the masonry.

4.—The track and rolling stock are supposed to be lighter.

All these reasons are invalid except those directly affected by the width, that is to say, the width of earthwork and the length of ties; and, theoretically, it is questionable if even these are. If the bridges, track, and rolling stock are lighter, they are correspondingly less efficient, and, generally, in lesser proportion, as, for instance, a 30-lb. rail will only carry one-quarter the load that a 60-lb. rail will.

In nearly all the discussions, etc., in regard to gauge, comparisons of costs are made between two or more lines already built, or the average cost of all the broad-gauge lines of a country is compared with the average cost of all the narrow-gauge lines. Such comparisons, of course, are valueless for the determination of the effect of gauge on cost of construction. The broad-gauge lines usually are of a much heavier type of construction, have heavier rails, heavier rolling stock, and more of it, expensive terminals, etc., as they usually do more business. To give only one example of the many which have been found in looking over previous discussions, the following,* is a very fair example of the kind of argument used and information on which arguments have often been based. Speaking of Indian railways, the statement was made that broad-gauge costs 2½ times as much as meter-gauge. If the Government of India had pinned its faith to broad-gauge it would now (1906) have 7 000 miles less of railroad. The cost of line in India having been £11 775 per mile for broad-gauge, as against £4 700 per mile for meter-gauge.

In the discussion following, it was pointed out that the broad-gauge lines did more business, had heavier work, that is, greater quantities of earthwork, larger terminals, etc., and that to take only one item, that of bridging and tunnels alone, these had cost Rs. 10 792 ($3 500) per mile for the broad-gauge as against Rs. 2 149 ($700) for the meter-gauge, so that if the meter-gauge lines had been built in the same location as the existing broad-gauge and equipped to do the same business (if this were possible) they would have cost quite as much.

In connection with the further discussion of this paper, the following estimates were made of the difference in cost of building broad- and narrow-gauge lines in India:

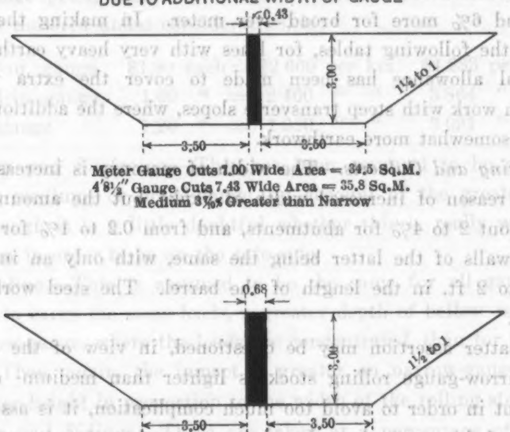
Sir John Hawkshaw............	£360	per mile	=	$1 800
Sir John Fowler...............	633	" "	=	3 165
Sir George Bruce..............	200	" "	=	1 000
Sir Guilford Molesworth........	425	" "	=	2 125
and in Victoria, Australia........	261	" "	=	1 305
and South Australia............	350	" "	=	1 750

It is to be noted that these differences are somewhat less than those shown in the estimates which follow, and are more nearly what they

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV.

might be if the calculations were made on a strictly theoretical basis, and including only the actual outlay for labor and material. In the estimates which follow (which are discussed in detail), not only has every reasonable allowance been made for any difference there is, but the same proportion is carried through the percentage of contingencies, interest during construction, etc., which tends to increase them, so that what difference they show is the maximum possible.

INCREASE IN EARTHWORK
DUE TO ADDITIONAL WIDTH OF GAUGE



Meter Gauge Cuts 7.00 Wide Area = 34.5 Sq.M.
5′6″ Gauge Cuts 7.68 Wide Area = 36.5 Sq.M.
Broad 5⅘% Greater than Narrow

FIG. 4.

The only satisfactory method of determining the effect of gauge on cost of construction is to calculate what effect a difference of gauge would have, assuming the same conditions.

We have already seen that, within the limits of practical railroading for the gauges under consideration, curvature and gradients are not affected by gauge, therefore the location would be the same for lines of either gauge through the same country, for, in order to make a fair comparison, we must assume the road designed in either case to do the same business.

Taking each item of cost, therefore, we find:

Right of Way, Station Grounds, Terminals, etc.—These are not affected, the difference in width of earthwork being only 0.70 m. between meter- and broad-gauge, so that the width of right of way is not changed. If the tracks are spaced closer in terminals for narrow-gauge, they must be longer to hold the same tonnage.

Clearing and Grubbing.—There is no difference.

Earthwork.—This item is the one most affected by gauge, but the proportion is very small, as will be seen by the diagram, Fig. 4, showing, in the case of a cut or fill 3 m. deep, 4% more for medium over meter and 6% more for broad over meter. In making the calculations in the following tables, for lines with very heavy earthwork, an additional allowance has been made to cover the extra width in mountain work with steep transverse slopes, where the additional width involves somewhat more earthwork.

Bridging and Culverts.—The width of masonry is increased somewhat by reason of increased width of banks, but the amount is very small, about 2 to 4% for abutments, and from 0.2 to 1% for culverts. the end walls of the latter being the same, with only an increase of from 1 to 2 ft. in the length of the barrel. The steel work will be the same.

The latter assertion may be questioned, in view of the fact that much narrow-gauge rolling stock is lighter than medium- or broad-gauge, but in order to avoid too much complication, it is assumed for purposes of this comparison that the same number of trains will handle the business, therefore the same weight of engine will be assumed. If any other assumption is made, that is, that heavier engines are used for the wider gauges, that means less trains and less cost of operation, which would compensate for the additional weight of the steel. The total cost of the actual bridging, omitting abutments, etc., is such a small proportion of the total cost of the railroad that any slight difference in this item would be very small.

Track.—Rails and fastenings will not be affected. The cost of rails is taken at $30 at Buenos Aires with about $150 per mile added for freight. The length of the cross-ties is affected directly as the width of the gauge, but as the function of the sleepers is to distribute the load, as well as to hold the rails in position, there would only be as many required for broad-gauge as would give the same bearing

area on the soil or ballast as those of the narrow-gauge, and, therefore, theoretically, there would be no difference in this item. Practically there is, however, but with the result that a better track is obtained. The number of the sleepers has been assumed, therefore, at 1 400, 1 500, and 1 600 per kilometer (2 254, 2 415, and 2 576 per mile) for broad-, medium-, and narrow-gauge, respectively.

Quebracho ties are used almost exclusively on the Argentine railways; they are very hard and durable, and, owing to their high cost, are spaced rather farther apart than is usual in the United States. The average price, including freight, at Santa Fé, a fairly central point, for the area under consideration, is:

5 ft. 6-in. gauge	\$1.90 each	= \$2 660 per km.	\$4 283 per mile.
4 ft. 8½-in. gauge	1.60 "	= 2 400 " "	3 864 " "
Meter-gauge	1.20 "	= 1 920 " "	3 091 " "

Laying and Surfacing.—This has been assumed to be somewhat more for medium- or broad-gauge, on account of the greater width, but, although it is a little doubtful whether this is really warranted, it is taken in order to be on the safe side.

Ballasting.—This is assumed to be the same for all gauges, for, in order to carry the same loads, a greater depth of ballast is required for narrow-gauge, where the loads are concentrated, than for the wider gauge. Then, again, the impact is greater on narrow-gauge, due to the greater height in proportion to the width of the rolling stock.

Yards and Sidings.—These are taken at a percentage of the cost of the track in each case. Here again, theoretically at least, there should be no increase in cost due to gauge, for, although the broad-gauge track costs more per linear foot, there is not so much of it required, broad-gauge trains of the same net tonnage requiring a lesser length of track than narrow-gauge trains. As, however, on lines of comparatively light or medium traffic the yards and sidings are likely to be made the same length in any case, the increase is made as shown.

Cattle Guards.—These will be slightly wider for wider gauge.

Miscellaneous Structures, etc.—Fences, Track Signs, Buildings of all kinds, Water Supply, Telegraphs, Docks and Wharves, General Expenses of Construction, Engineering and Legal Expenses, would not be changed.

Contingencies, Expenses of Promotion, Interest During Construction.—These are figured at percentages of the totals.

Equipment.—For the same amount of traffic, the equipment would probably cost slightly less for the wider gauge on account of the larger capacity of the cars, but, assuming as nearly as possible the same weight of engines, etc., this would not be a great deal.

Tables 15, 16 and 17 show the differences due to gauge alone on lines of light, medium, and heavy earthwork and bridging, and assuming the same traffic and carrying capacity on all. It will be noted that for lines of light and medium construction there is very little difference in the percentage of increase, what difference there is being a less percentage where the work is heavier, medium-gauge averaging 6% more than narrow-gauge and broad-gauge 10% more than narrow. For very heavy lines, that is, mountain lines with steep transverse slopes, there should be some additional increase in earthwork due to the increased width. This has been allowed for, and in this class of lines the medium-gauge might be assumed to cost 11% more and the broad 16% more than the narrow.

The summary of the totals is given in Table 14, in which the percentage is the increase above the cost of narrow-gauge.

TABLE 14.—SUMMARY OF COSTS.

Gauge.	Cost per Mile, in Gold Dollars.			Cost per Kilometer, in Gold Dollars.		
	Light.	Medium.	Heavy.	Light.	Medium.	Heavy.
Narrow........	$29 772	$54 760	$92 573	$18 458	$33 951	$57 395
Medium.......	31 581 + 6%	57 920 + 6%	102 892 + 11%	19 580	35 910	63 793
Broad.........	32 861 + 10%	60 430 + 10%	107 553 + 16%	20 874	37 467	66 683

It may be noted that the assumed increases in Table 14, which the writer believes to be the maximum or outside amounts, are not more than the usual allowances for contingencies in work of this magnitude.

Cost of Conversion.

The following estimate of the cost of changing the gauge, although not based on detailed surveys or calculation of quantities, is believed to represent average conditions, based on observations by the writer after an inspection of practically all the meter and standard-gauge

lines of the Argentine, as well as of a considerable proportion of the broad-gauge lines.

It is believed that it can be safely assumed that the cost of conversion, including a liberal allowance for rolling stock, will not exceed the figures given, and will probably be less. It is to be noted, however, that these estimates do not include any betterments, such as heavier rails, renewing bridges, etc., or any other works of improvement which it might be desirable to carry out at the time the gauge would be changed; but they do include an allowance for all new sleepers. It may also be noted that most of the lines which would be changed first are through easy country, and the cost would be less than the average.

TABLE 15.—COST OF NEW CONSTRUCTION.

LIGHT EARTHWORK AND BRIDGING.

	Cost per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½-in.	5 ft. 6-in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	200	200	200
Earthwork	4 000	4 310	4 570
Bridges	2 000	2 150	2 300
Culverts	2 000	2 150	2 300
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 030
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply, etc.	2 000	2 000	2 000
Telegraph and telephone	500	500	500
Docks and wharves	50	50	50
General expenses and administration	300	300	300
Engineering	1 000	1 000	1 000
Legal expenses	250	250	250
	$23 280	$24 940	$26 140
Contingencies, 5%; Expenses of promotion, 2%; Interest during construction, 8% } 15%	3 492	3 741	3 921
	$26 772	$28 681	$30 061
Equipment (rolling stock, etc.) assumed for same amount of traffic	3 000	2 900	2 800
	$29 772	$31 581	$32 861
Per kilometer	$18 458	$19 580	$20 374

TABLE 16.—COST OF NEW CONSTRUCTION. MEDIUM EARTHWORK AND BRIDGING.

	Cost per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½-in.	5 ft. 6-in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	250	250	250
Earthwork	18 450	19 430	20 260
Bridges	5 000	5 500	6 000
Culverts	5 000	5 300	5 600
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 030
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply, etc	2 500	2 500	2 500
Telegraph and telephone	550	550	550
Docks and wharves	50	50	50
General expenses and administration	400	400	400
Engineering	1 300	1 300	1 300
Legal expenses	270	270	270
	$44 750	$47 580	$49 850
Contingencies, 5%; Expenses of promotion, 2%; Interest during construction, 8% } 15%	6 710	7 140	7 480
	$51 460	$54 720	$57 330
Equipment (rolling stock, etc.) assumed for same amount of traffic	3 300	3 200	3 100
	$54 760	$57 920	$60 430
Per kilometer	$33 951	$35 910	$37 467

The following* shows the actual cost of changing certain lines in India from meter to 5 ft. 6 in. The costs include rolling stock and, in at least one case, the Salt Branch of the North Western, the cost of changing from 41-lb. to 75-lb. rails, and in the last case there were "heavy charges for bridge renewals and heavy permanent way".

COST OF CONVERTING METER-GAUGE TO 5 FT. 6-IN. IN INDIA.

			Per mile.	Per kilometer.
Salt Branch of North Western,	50.0	miles	£2 867 = $14 350 =	$8 600.
Nagpur-Chattisgarh,	145.2	"	£2 583 = $12 915 =	7 750.
Kotkapura-Firozur,	27.9	"	£1 472 = $73 60 =	4 415.
Gudur to Nellore,	24.2	"	£4 125 = $20 625 =	12 374.

* From *Minutes of Proceedings*, Inst. C. E., 1906.

The author of the paper, Sir Frederick Robert Upcott, who gave these costs, estimated that the cost of changing the existing meter-gauge lines of India to 5 ft. 6-in. would be £2 500 ($12 500) per mile, or about £1 500 ($7 500) per kilometer.

TABLE 17.—COST OF NEW CONSTRUCTION.
HEAVY EARTHWORK AND BRIDGING.

	Cost per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½-in.	5 ft. 6-in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	300	300	300
Earthwork	39 490	44 500	46 000
Bridges	10 000	12 000	13 000
Culverts	10 000	11 000	12 000
Track; Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 030
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply, etc	3 000	3 000	3 000
Telegraph and telephone	600	600	600
Docks and wharves	50	50	50
General expenses and administration	500	500	500
Engineering	1 800	1 800	1 800
Legal expenses	300	300	300
	$77 020	$86 080	$90 220
Contingencies, 5%; Expenses of promotion, 2%; Interest during construction, 8% } 15%	11 553	12 912	13 533
	$88 573	$98 992	$103 753
Equipment (rolling stock, etc.) assumed for same amount of traffic	4 000	3 900	3 800
	$92 573	$102 892	$107 553
Per kilometer	$57 395	$63 793	$66 683

It is stated* that the cost of changing the gauge of the lines in Queensland and West Australia from 3 ft. 6-in. to 4 ft. 8½-in. will be $111 000 000, or an average of $16 600 per mile ($10 000 per km.) for the 6 662 miles to be changed.

It has also been stated that the estimated cost of changing the gauge of the railways in Japan will be some $125 000 000 for about

* *Railway Age Gazette*, July 11th, 1913.

5 000 miles, or $25 000 per mile ($15 000 per km.). This seems high, but, of course, the topography of the country is quite rugged and mountainous and the railways have many large bridges and tunnels.

TABLE 18.—Estimate of Cost of Changing 1 Mile of Meter-Gauge to Medium- or Broad-Gauge.

Assuming Average Conditions on the Meter-Gauge Lines of the Argentine.

		Gold Dollars.	
		Medium.	Broad.
Right of way, station grounds, terminals, etc., $10 000 for additional terminal land for every 300 miles		$33	$33
Earthwork: Say, 5% additional for medium and 7% for broad on assumed present average cost of about $20 000 per mile		1 000	1 400
Bridges and culverts: lengthening		2 000	2 500
Track: Medium gauge:			
2 400 new sleepers at $1.60	$3 840		
New fastenings	250		
Taking up, relaying and surfacing	1 500		
		5 590	
Broad-gauge:			
2 600 new sleepers at $1.90	$4 940		
New fastenings	250		
Taking up, relaying, and surfacing	1 500		
			6 690
Yards and sidings: 20% of main line		1 118	1 338
Cattle guards, nominal		15	15
Fences, track signs, etc., no change			
Buildings, shops, stations, section houses, turntables, water supply. Will only involve changing tracks and spacing in shops, turntables, engine-houses, and running sheds, say $200 000 for every 300 miles		630	700
Signals and interlocking		75	75
Telegraph and telephone, no change			
General expenses and administration		100	100
Engineering		250	250
Legal expenses		25	25
Interest during construction		600	700
Contingencies		600	700
		$12 036	$14 526
New rolling stock, say $5 000 per mile. Credit old rolling stock 3 500 per mile		1 400	1 600
Per mile		$13 436	$16 126
Per kilometer		8 330	9 998

Cost of Operation.

Various statements have been made in regard to the relative costs of operation and maintenance of lines of different gauges, almost invariably however, so far as the writer has been able to discover, based on comparisons of the costs on two existing lines or groups of lines. Such comparisons are not of great value, for no matter how generally similar two lines may be, or seem to be, the inherent differences in

physical condition, amount and character of traffic, rolling stock, management, policy, etc., will be considerable. An attempt was made by the writer to analyze the details of the operating costs of certain lines, and from this to form some idea of the actual effect on each item of a change or difference in the gauge, but it was found that there was so little information available that the result would be too much in the nature of a pure guess to be of any value, and as there seems to be no really good way of determining what actual effect a difference of gauge would have on the operation and maintenance costs, such comparisons between the results on lines of different gauges in various parts of the world as are available are given for what they are worth, followed by some discussion of the effect of gauge on maintenance of permanent way and an analysis of the possible economies by the use of heavier rolling stock, etc., both in maintenance of equipment and costs of operation.

Indian Railways.—In regard to the railways of India, the following statement was recently made by Sir Guilford Molesworth, showing that although the train-mile costs on the narrow-gauge lines of that country were lower than on the broad, the net ton-mile costs were higher.

Taking the East India Railways (5 ft. 6-in.) as 100, the costs on the following narrow-gauge lines were:

	Per train-mile.	Per ton-mile.
Assam Railway	71	315
South Maharatta	56	280
Burma Line	64	290

A statement prepared by Sir Frederick Upcott, giving the comparative statistics of eight of the principal broad-gauge and eight of the principal narrow-gauge lines, for 1903, showed that, though the earnings on the narrow-gauge lines gave practically the same returns on the capital invested as on the broad-, the average charges were:

	Cents, in gold. Broad.	Narrow.
Per passenger-mile	0.42	0.40
Per freight-mile	0.92	1.14

and the cost of carriage of freight per ton-mile was:

0.5 cent for the broad-gauge.

0.6 " " " narrow-gauge.

It is interesting to note, here, that the average weight of trains on the broad-gauge was 167 tons, as compared with 88 tons for the narrow-gauge.

Australian Railways.—The ton-mile costs in Australia are not available, but the train-mile costs for 1910-11 were as follows:

Broad	$1.20
Medium	1.09
Narrow	1.32

These figures, taken together with the fact that the net earnings per mile of the medium-gauge lines were double those of the narrow-gauge, indicate the inferiority of the earning power of the latter.

Argentine Railways.—In the Argentine the average operating costs of all lines for 1909 were as follows:

	Broad.	Medium.	Narrow.	Broad.	Medium.	Narrow.
	Per train-kilometer.			Per train-mile.		
Dollars, gold. . .	$1.006	$0.755	$0.876	$1.620	$1.216	$1.410
	Per ton-kilometer.			Per ton-mile.		
Cents, gold......	0.928	0.957	0.967	1.494	1.540	1.557

The ton-mile (ton-kilometer) used for these figures is that of the "peso util" or "paying weight" transported, and includes passengers, with ordinary baggage, calculated at 100 kg. (220 lb.) each, excess baggage, and express, as well as freight; but, as the proportion of revenue from passengers, baggage, and express is much greater on the broad-gauge lines, being 31% of the total as compared with 19% for the narrow-gauge, the comparison is hardly fair to the broad-gauge. It shows, however, the same tendency to low cost per ton-mile for the wide-gauge, although the train-mile costs, due to the heavier trains hauled, are higher.

It is to be noted, as previously stated, that the medium-gauge lines, do not offer any real comparison on account of local conditions, and this applies also to the ton-mile costs on these lines, which are given later.

Taking the figures as they stand, and using the number of ton-kilometers for 1909, they show a saving in operating costs of a little more than $100, gold, per kilometer ($160 per mile) per annum in favor of the broad-gauge, which, of course, is not very large. In discussing these costs, however, consideration must be given to the essential difference and physical characteristics of the broad- and narrow-

gauge lines at this time (1909). The narrow-gauge lines of the Argentine were then doing a fair amount of business, as more or less local lines, that is, none of them reached the National Capital nor did they partake of the character of trunk lines. The Central Cordoba was then building to Buenos Aires, and now (1911) all the narrow-gauge lines realize that, if they are not to be entirely crowded out, they must get in close touch with the National Capital in order to stay in business, and spend large sums for capital account to put their lines in condition even to meet reasonably the competition existing or to be expected. The narrow-gauge lines were then capitalized at about \$28 000, gold, per km., as compared with \$42 000, gold, for the broad-gauge, but they are now faced with the necessity of capital expenditure which will bring them well up toward the capitalized value of the broad-gauge. A large part of the capital of the latter is invested in the expensive Buenos Aires terminals, and other facilities at all points on their lines, which they have found it necessary to have in order to do their business, which the narrow-gauge lines have yet to acquire and develop, and on the capital expenditure for which they must earn the interest. It is also quite certain that many of the narrow-gauge lines would have to be double-tracked in order to handle the business.

If it could be assumed that the foregoing figures would hold good for any amount of business to be hauled, they would not present a very formidable argument for the broad- as compared with the narrow-gauge, but all railroads, as well as all other enterprises in any country as progressive as the Argentine, must grow with the country or be crowded out, and this is what is beginning to happen to the narrow-gauge lines. The business coming to them is more than they can handle economically with the plant they have, because it tends to cause congestion.

The need of adequate facilities for doing business and the increased cost of operation due to their lack when the amount of business offered passes a certain limit, was shown in 1906 in the United States, when many of the railroads of the country suffered serious losses because of the congestion caused by the large amount of freight offered for transportation which they had not the facilities for handling.

The real test, of course, is the final net earnings on the investment, and here the broad-gauge lines, in spite of their heavier capitalization,

are a long way ahead, with average earnings of 5.66% on the capital invested, as compared with 2.41% earned on the capital invested in the narrow-gauge lines, so that it is evident that the ton-mile figures do not tell the whole story.

Taking freight alone, on the basis of these 1909 figures, the receipts per ton-mile make a still better showing in favor of the narrow-gauge, the averages being:

	Freight receipts. Ton-kilometer.	Ton-mile.	
Broad,	1.17	1.95	cents, gold.
Medium,	1.26	2.10	" "
Narrow,	1.05	1.75	" "
Average for all lines,	1.15	1.92	cents, gold.

It may be noted, also, that the rates on the Central Cordoba and Santa Fé lines are the lowest of any lines in the country, and it is these which tend to lower the average for the narrow-gauge lines, but it is believed that the explanation given above shows the reason for the apparent cheaper cost of operation, or, rather, lesser rates on these two narrow-gauge lines. It may also be added that the property of neither has been kept in first-class physical condition, and, as already stated, their net earnings show a much lower percentage of the capital invested than do the broad-gauge lines. The Government lines, also, which are narrow-gauge, are operated at a loss. If the freight charges on these lines were fixed to give the same return on the capital invested they would, of course, be much higher.

Comparison of Ton-Mile Costs.—The average receipts per ton-mile of the Argentine railways in 1909 are given above. The average for the United States in 1912 was 0.85 cents. On 70% of all the railroads the average was less than 1.0 cent per ton-mile, and the lowest average rate was 0.48 cent per ton-mile.

The actual cost of moving freight on the Bessemer and Lake Erie Railway in 1909 was stated to be* 0.230 cent per ton-mile; this included terminal charges on a road only 144 miles long; adding interest on the investment would bring the total cost to 0.280 cent per ton-mile.

Maintenance of Permanent Way.—In regard to maintenance, it seems quite evident that this would be considerably more on narrow-

* *Engineering News*, April 21st, 1910.

gauge lines having any large amount of traffic, and especially if it was desired to run passenger trains even at reasonably fast speeds.

If it were possible to build and maintain a track without inequalities, or if the inequalities of the line and the forces tending to produce oscillation could be reduced in the same proportion as the width of the gauge, the cost of maintenance might be considered as proportional to the width, but these conditions are unattainable.

The narrower the gauge the greater is the angle through which the vehicle is canted laterally, through a certain depression or elevation of one of the rails, and the greater, therefore, is the inequality produced in the loads on the springs on opposite sides, and, consequently, on the two rails also, so that the narrower the gauge the greater the extent of lateral oscillation to which any given inequality in the line will give rise, and this is a point of especial importance in districts where, from the variations of climate or other influences, the permanent way is likely at times to get more or less out of repair.*

It is to be noted, also, that the use of stone ballast is almost indispensable on narrow-gauge lines, in order to have track on which speeds of more that 30 miles per hour are safe, and that the depth of the ballast must be increased in order to distribute properly the loads which, in the narrow-gauge, are concentrated on a smaller area.

A few years ago it was quite generally thought that the principal function of the ballast was to afford proper drainage for the track, though even as long ago as 1887-90 experiments, which were conducted in Germany by Railroad Director Schubert, had indicated the value and necessity of ballast for the proper distribution of the loads. In view of the great increase in axle loads in recent years, an extended series of experiments was made in 1908-10 by the Pennsylvania Railroad to obtain further light on this phase of the subject.† These experiments showed quite clearly and conclusively the part played by ballast in the distribution of the loads, and the necessity of a sufficient depth to take care of this. It is quite certain, therefore, that the concentration of loads on lines of narrow-gauge, both by reason of the smaller bearing area of the sleepers and the additional impact due to the higher center of gravity and greater oscillation, must be taken care

* See also p. 327 *et seq.*

† *Proceedings*, Am. Ry. Eng. Assoc., 1912.

of by an increase in depth of ballast, if equally good results are to be obtained.

Maintenance of Equipment.—It is quite difficult, if not impossible, to make any estimate of the effect of gauge on the cost of maintenance of equipment. It is quite sure that the cost of maintenance of both locomotives and wagons is not proportionate to either their size or capacity, that is to say, it does not cost twice as much per annum for the maintenance of a 200-ton engine as it does to maintain a 100-ton engine; two 25-ton cars will cost more to maintain than one 50-ton car doing the same work. The passenger equipment for narrow-gauge has only about three-quarters of the carrying capacity of the broad-gauge for vehicles of the same length, so that, to do the same amount of business, there will be a larger number of vehicles, larger proportion of dead weight per passenger, and, therefore, a larger cost of maintenance per revenue unit.

If it could be assumed that all or even the greater proportion of the business of the Argentine, if carried on broad-gauge lines, could be handled by locomotives and in freight cars of twice the capacity of those now in general use on the narrow-gauge, one could say at once that the cost of maintenance of equipment would be greatly lessened thereby. This, however, is not the case, and there are many locomotives and cars in use, and of a useful size, on the narrow-gauge, equal in capacity to many on the broad-, and it is only a certain proportion of the business that can be handled to better advantage in cars and by locomotives of larger capacity.

Taken altogether—passenger cars, freight cars, and locomotives—it seems quite sure than the cost of maintenance of equipment would be more on narrow-gauge lines than on the broad-gauge, by reason of the larger number of vehicles required to handle the same amount of business. Just how much this item would amount to it is not possible to say, though the figures in Table 19 may give some idea.

Effect of Larger Rolling Stock Units on Costs of General Repairs.—Assuming quite conservatively that half the business could be handled on the broad-gauge by equipment having 50% greater capacity than that now in use or possible on the narrow-gauge, then the number of units of rolling-stock would be reduced one-sixth. The cost of repairs on the larger units would be somewhat greater, but not propor-

TABLE 19.—COST OF MAINTENANCE OF LOCOMOTIVES AND CARS IN 1909 ON THE NARROW-GAUGE LINES OF THE ARGENTINE.

	Number in service.	Repairs and Maintenance.	
		Total cost.	Cost, each.
Locomotives	500	$1 051 332	$2 102
Passenger coaches	599	258 477	430
Baggage cars	402		
Freight cars	13 540	683 175	47
Service cars	540		
		$1 992 984	

tionately so, say, roughly, only 50% of the increase. That is, for example, taking the case of the locomotives, 250 would remain as at present, each costing $2 100 a year for repairs, the other 250 would be replaced by two-thirds of the number, each unit having 50% greater capacity and costing 25% more for repairs, that is, there would be 167 each costing $2 625 for repairs per annum. The total cost of repairs would then be:

Locomotives	250 × $2 102 =	$252 500
	167 × 2 625 =	438 375
Passenger coaches	300 × 430 =	129 000
	200 × 537 =	107 400
Freight cars, etc.	7 240 × 47 =	340 280
	4 826 × 59 =	284 734
		$1 825 289

This shows a saving of $167 695 per annum, equal to about $23 per km. of line, with the business of 1909. With a live policy of operation, the building of branches to develop the existing main lines, it is not unreasonable to assume that, by 1919, say in 10 years, the business will be increased to three times the present amount, and a large proportion of it will need to be handled in heavy trains.

There would be all the new mileage on which there would be a similar saving to that just shown, the new mileage would bring as much business to the old lines as these now have, and the natural increase in the country adjoining the old lines might be at least equal to the business now, so that on this item of repairs alone there would be a saving of from $70 to $100 per km. on the present lines, and at

least \$20 to \$30 on the new lines, or, say, an average of \$50 per km. per annum on the whole system then in operation.

It is to be noted that if the change of gauge is brought to pass, the new equipment, if properly selected, would show a much smaller cost of repairs, as it would be better adapted to the requirements of the service. There would be also, without going to extremes, much of it of double the average capacity of the present narrow-gauge rolling stock, instead of only 50% greater as just assumed, so that the foregoing estimate is quite conservative.

Effect of Small or Large Cars on Train Resistance, Loading, Etc.— The effect of small cars on train resistance has already been pointed out.* Their effect on the cost of operation is indicated by the following:

Assume an engine of the Mikado (2-8-2) type:

Total weight of engine and tender....	246 600 lb.
Weight on drivers................	110 500 "
Maximum tractive power..........	28 900 "

This is about as large an engine as can be built for meter-gauge of a type which will give large power capacity with ability to make fairly good speeds with heavy trains.

Engines of the Mallet or articulated type are built of much greater power, but, as that type only operates efficiently at very low speeds and is best adapted to sections of heavy gradients, it does not afford as good a basis for comparison for general conditions, in fairly easy country, similar to that found in the Argentine, where much of the freight must be moved quickly, as does the Mikado type.

The tonnage-rating formula of the American Railway Engineering Association is

$$R = 2.22\,T + 121.6\,C.$$

in which

R = Total resistance on level tangent, in pounds,

T = Total weight of train behind the tender, in tons,

C = Number of cars.

This assumes good, fair rolling stock and track, and that the resistance does not increase at ordinary freight train speeds between 7 and 35 miles per hour.†

* p. 351.

† *Proceedings*, Am. Ry. Eng. Assoc., 1910.

Using this formula and applying it to trains of fully-loaded cars on a line with grades of 0.6%, it is found that the engine assumed, when working to full capacity (*i. e.*, exerting the same power in each case), can haul the loads shown in Table 20.

TABLE 20.

	No. of cars.	Net tons per car.	Tare per car, in tons.	Gross tons.	Net tons of freight.
Broad-gauge cars	24	50	18	1 632	1 200
Standard-gauge cars	24	50	18	1 632	1 200
Meter-gauge cars	34	33	13	1 564	1 122

This is an increase of 78 net tons per train, as between meter- and standard-gauge. The formula deduced by Professor Schmidt, of the University of Illinois, from the results of extensive experiments made to determine the influence of car weight on train resistance, shows quite similar results, though even less favorable to the lighter cars.

The average receipts per ton of freight per kilometer in 1909 were 1.15 cents, gold. For the train under consideration, therefore, the average increased receipts for the same operating costs per train-kilometer would be 89 cents, gold, or, with an average goods traffic of one train daily each way, $1.78 per km. per day, or about $500, gold, per km. ($830 per mile) per annum.

It is stated that some of the traffic in the Argentine can be handled better in cars of small capacity. Assuming that half the business is handled in small cars on which there is no saving, there would still be an increased income of $250, gold, per km. per annum, due to the use of the larger cars.

Assuming, then, that an average kilometer of broad-gauge line costs from $3 000 to $5 000, gold, more to build than meter-gauge, the investment is warranted from this standpoint alone, and, taking the figures at the end of the paper, the change of gauge of the existing lines would also be warranted.

The foregoing takes no account of the further saving which may be effected on the wider gauge by the use of larger engines than are at all possible on meter-gauge, thereby further increasing the size and weight of trains and, consequently, obtaining lower operating costs, nor the fact that the speed of passenger trains is limited on the narrow-

gauge, both on account of the size of the engines and the lesser stability of trains.

Economy in Increased Train Loads by Reduction in Number of Trains.—The possibilities of the use of the larger cars and heavier engines depends, of course, to a large extent, on the kind of traffic to be handled. Such articles as cereals, cattle, timber and other forest products can all be handled to better advantage in large cars. The tonnage of these articles handled on the narrow- and medium-gauge lines of the Argentine in 1909 is shown in Table 21.

TABLE 21.

	Narrow.	Medium.
Cattle	219 631	232 680
Wool, hides, etc.	40 864	38 278
Cereals	955 448	294 659
Sugar	223 789	3 528
Minerals	35 500	....
Construction material*	2 153 312	140 360
Firewood	768 828	30 045
Charcoal	302 881	33 495
	4 700 253	778 045

* Includes forest products.

That is, about 5 500 000 tons, out of a total of 8 500 000 tons, were handled on these lines, or 65% of the total. The average haul on all freight in 1909 was about 180 km., and the total length of lines about 9 000 km. It may be noted, also, that with the opening of the through connection to Paraguay and the extension of a railway line through Missiones, the shipment of timber suitable for building purposes from these northern points to the south is likely to assume large proportions, and this will all be long-haul business which must be handled cheaply to make it move.

The Mikado engine previously referred to can haul, say, 1 000 tons of net load on 0.6% grades. The largest engine yet built for narrow-gauge, the Mallet (articulated) engine previously referred to, might haul, say, 2 000 tons net load. There are, however, on some parts of these lines, especially in the southern section of the Central Cordoba and all through the Provinces of Entre Rios and Corrientes, grades of 1.0% or heavier. Taken altogether, therefore, considering the grades, the fact that the big articulated engine is not a useful type for

general use, and that a certain number of engines of the Mikado type must be kept in use for general freight services, it seems safe to assume that, even under the best of conditions on the narrow-gauge lines, the heavy trains will not exceed an average capacity of 1 500 net tons, whereas on the broad- or medium-gauge this could be doubled.

The operation of such extremely heavy trains, however, is not a practical proposition in the Argentine at present, nor is it likely to be in the immediate future, though the increasing distances to the new areas constantly being opened up will continually increase the tendency in this direction.

In referring to trains of this weight, it must not be assumed that the very much less average weight of trains has been lost sight of, the average net train loads in the Argentine in 1909 having been:

Broad	327 tons.
Medium	198 "
Narrow	211 "
All lines.......	293 "

The average train loads in the United States in 1910 for all lines was 380 tons, and in Group II, comprising the States of New York, Pennsylvania, and New Jersey, 502 tons, yet there are records of single trains of more than 8 000 tons gross, and the great reduction in the cost of transportation, which has been obtained in the United States in spite of the high cost of labor, has been brought about by the use of heavy trains for bulk traffic which can only be moved profitably at low rates.*

For the purpose of this argument, therefore, and to get some idea of the possible savings due to the reduction in the number of trains, it is entirely reasonable to assume that the practical train loads for the broad-gauge may easily be one-third greater than on the narrow-gauge, or say, 2 000 net tons, as compared with the possible maximum of 1 500 tons on the narrow-gauge.

Taking into consideration the growth of the country and a continuance of the railway development, it might be assumed that within a few years the amount of this class of traffic on the lines under consideration will be doubled to, say, 12 000 000 net tons. On the medium-gauge, with train loads of 2 000 net tons, this would mean 6 000 trains,

* See also p. 342.

and the narrow-gauge would require 8 000 trains. The average haul on this traffic is now 180 km., and the length of haul is practically certain to increase. An average engine division is about 150 km., and, considering the almost certain increased length of haul, one should add, say, one-third more to the number of trains (that is, train runs handled by one crew), making the numbers 8 000 and 10 667.

The present actual operating cost, that is, cost of coal, lubricants, train crews, repairs to rolling stock, etc., is about 50 cents, gold, per train-kilometer, of which about one-third is for fuel and water. This, however, is with trains very much lighter than those under consideration. With heavier trains the fuel and water would be increased, approximately, directly in proportion to the weight of the train. Repairs would be affected somewhat more, but not in proportion.

The average present train load to which these figures apply is about 500 tons. So taking this cost of 50 cents, of which, say, 15 cents is for fuel, water, and lubricants, and multiplying these items by three, and increasing the remainder by 25%, gives the following cost per kilometer per train of 1 500 tons:

Fuel, etc..... 15 cents × 3 = 45 cents.
Other items.. 35 " + 25% = 44 "
Total................. 89 cents per train-kilometer.

In increasing the weight of the train from 1 500 to 2 000 tons, the other items would not be affected as much as by the increase as above from 500 to 1 500. Fuel might be increased 33%, though this is doubtful,* and the other items hardly at all, but, say, 10 per cent. We have, therefore, for a train of 200 tons:

Fuel, etc..... 45 cents + 33% = 60 cents.
Other items... 44 " + 10% = 48 "
Total................. 108 cents per train-kilometer.

Taking the average run as 150 km., the cost per train would then be:

For trains of 1 500 tons, 150 km. × 89 cents = $133.5 per train.
" " " 2 000 " 150 " × 108 " = 162.0 " "

and there would be:

On the narrow-gauge 10 667 trains at $133.5 = $1 423 912
" " medium-gauge 8 000 " " 162.0 = 1 296 000
$127 912

* See p. 345 et seq.

or equal to about $14.20 per km. of line, due to the reduction in the number of trains.

In the foregoing calculation it has been assumed that the costs of fuel, water, and lubricants increase directly in proportion to the weight of trains, but most of the larger modern engines have shown a marked economy in this respect, and the coal consumption, etc., has been quite a little less, proportionately, to the work done.

It may be noted that, in making these assumptions, it is not considered that there will be any additional costs on the standard-gauge, due to the operation of the heavier trains, which in some cases have to be considered. It is believed that a 1 500-ton train will be quite as destructive to the track on the narrow-gauge as a 2 000-ton train on the medium-gauge, and that the time factor will not enter into the question, as an engine designed to haul a 2 000-ton train on standard-gauge will make as good time and with less effort than will a narrow-gauge engine designed to haul 1 500 tons, on the same grades and alignment.

The Necessity of an Efficient Transportation Machine.—The possibility of the competition of other broad-gauge lines or of river transportation has been referred to, and it must be kept constantly in mind that cheap, but at the same time efficient, transportation is a necessity, both for the development of the country and on account of the river competition.

The following quotation from a recent review of the general railroad situation in the United States by M. C. Colson, an eminent French engineer, shows how cheap railroad service has affected transport by water, and has a direct bearing on the necessity of establishing a railway system in the Argentine, which will accomplish similar results there. It hardly seems possible with the narrow-gauge to approach the low cost of transporting bulk freight which can be achieved on the medium- or broad-gauge.

After pointing out that the mileage of railways in the United States amounts to 10% more than that of all the railways of Europe, though the population is only 90 000 000, as compared with 450 000 000 in Europe, he states:

"Before the railways [of the United States] became developed, attempts were made to extend the inland waterways by constructing many canals, but inland navigation gradually died out, except on the Great Lakes, where it has a quasi-maritime character, and on some

exceptionally placed waterways, as soon as a better sytem of transport became known. It was the railway which made it possible to develop with unexampled rapidity this great continent, to develop farms whose produce, sent to the Old World at very low prices, there produced the agricultural crisis of thirty years ago. Then it helped to extend a population there which will very soon consume all the produce of its own country, so that its growth is an important factor in the general rise in prices, which is now producing with us a crisis in an inverse sense of the former one, namely: the dearness of food. Finally, the railways, which carried coal and minerals at rates even lower than those charged for cereals, helped to give the United States an industrial development, the general growth and magnitude of which are just as surprising as the former development of farming."

Statistics are then given showing in detail the much lower costs of transportation in the United States as compared with those of Europe,* and he continues:

"In Europe 10-ton wagons are still the rule, 20-ton wagons are exceptional, and 40-ton wagons are very rare. In the United States, out of 2 400 000 wagons, there are only 12 000 which take less than 18 tons, and most can take 18 to 33 tons; 634 000 can take 33 tons, and 390 000 can take 42 to 54 tons. These working conditions, which have resulted in the historical development of American farming and industries, make it possible to have very low rates and still work at a profit."

Summary.

The preceding discussion may be briefly summarized as follows:

Curvature.—Within the limits of the practical operation of railroads, a curvature of any radius which is feasible on meter-gauge is as practical on 4 ft. 8½-in., or even 5 ft. 6-in.

Gradients.—The working of trains on steep gradients is not affected by gauge, except in so far as the narrow-gauge cuts down the locomotive capacity, and is, therefore, at a great disadvantage.

Location.—In view of the foregoing, the narrow-gauge (3 ft. 0-in. to 3 ft. 6-in.) does not permit better adaptability of the alignment to the conformation of the ground.

Cost of Construction.—This is so little more for 4 ft. 8½-in. as to be more than counterbalanced by economy in operation.

Speed.—This is an important feature in railway operation which is adversely affected by narrow-gauge.

* See also the statement on p. 342.

Stability.—The relative stability of narrow-gauge trains in motion is much less than that of trains on medium- or broad-gauge; consequently, high speeds are impractical and may be dangerous.

Track Stresses.—Much higher track stresses are produced on the narrow-gauge; there is greater impact and therefore more difficulty in maintaining track on narrow- than on broad-gauge.

Rolling Stock Cars.—The greater width of narrow-gauge cars in proportion to the gauge is considered a necessary evil rather than an advantage, and it is shown that there is probably no advantage to be gained by making the broad-gauge cars any wider than they are at present. In the matter of passenger cars, the narrow-gauge is at a decided disadvantage, both as to cost per unit of accommodation and in dead weight per passenger. In freight cars, there is little difference in cost, within the limits of size possible on the narrow-gauge, though cars of twice the capacity are available for medium- or broad-gauge. The increase in weight of modern freight trains, in order to decrease the cost of operation, has made heavier draft rigging necessary, thus tending to increase the proportion of dead to paying load to a much greater extent on small capacity cars than on the larger ones, than has been the case heretofore.

Locomotives.—The cost per unit of capacity of locomotives is about the same for each gauge up to the limit of the narrow-gauge, which limit for all types is about half of that of the 4 ft. 8½-in. or 5 ft. 6-in. There is an economy in the use of heavier types of locomotives not available for the narrow-gauge, and the use of cars of large capacity is a necessity for cheap transportation.

Train Resistance.—Train resistance is decreased by the use of larger units.

Capacity.—Figures are given tending to show that the relative capacity of narrow-gauge lines is not more than half that of medium- or broad-gauge.

Heavier Engines and Trains.—There is a general discussion showing the part played by increase of train loads in reducing costs in the United States.

Cost of Operation.—The ton-mile costs, on both Indian and Australian Railways, as well as those in the Argentine, are less on the medium- and broad-gauge than on the narrow-gauge, although the train-mile costs are often higher.

The lowest ton-mile costs on any railway in the Argentine are more than twice as high as any in the United States.

The saving in cost of operation, due to the use of larger cars and heavier trains, is considerable, and would in itself go a long way toward meeting the interest charge on the additional cost of the wider gauge.

Cost of Maintenance.—Maintenance of way probably costs more per traffic unit on narrow- than on medium- or broad-gauge on any but lines of very light traffic.

Maintenance of equipment should be less on medium- or broad-gauge by reason of the larger capacity, and, consequently, lesser number, of locomotives and vehicles, provided there is business to warrant their use.

Need of Efficient Transportation Machine.—The necessity of an efficient transportation machine to develop the country properly is pointed out, as well as the part played by the railways in developing the United States, the country most nearly comparable with Brazil and the Argentine.

THE MOST SUITABLE GAUGE.

We now come to the consideration of the most suitable gauge for the development of these countries. Such details of the topography, general conditions, traffic, etc., as are necessary for a proper understanding of the requirements are given in Appendix B.

In the Argentine, as in India and Australia—the only other countries which have large areas, large traffic, and different gauges—whatever the cause may be, however different any two lines may be, it is a fact that the greatest development has been along the lines of the broad- or medium-gauge railroads, and the big business is done by them. This is shown in Table 22.

TABLE 22.—RECEIPTS, IN GOLD DOLLARS, PER MILE OF RAILWAY PER ANNUM.

Country.	Year.	Gross Receipts.			Net Receipts.		
		Broad.	Medium.	Narrow.	Broad.	Medium.	Narrow.
Argentine	1911	$7 885	$3 450	$3 905	$2 340	$1 395	$1 000
India	1906	9 870	....	3 910	4 845	....	2 090
* Australia, Government	1911	6 985	3 135	3 515	2 565	3 165	1 190
Australia, Private	1911	2 040	3 850	2 510	1 000	3 055	1 150

* Leaving out South Australia, total receipts $26 700 for two gauges not divided; and Northern Territory, $1 875 narrow-gauge, too small to be averaged with others.

It is difficult to get close figures for Brazil, but for 1906—the latest for which official statistics are published—the three systems which contain among them the broad-gauge lines, namely, The Central of Brazil, The Paulista, and the São Paulo Railways, operate (counting both broad- and narrow-gauge) less than 15% of the total mileage of the country, and their receipts amount to nearly 65% of the total, averaging about $12 000 per mile of line.

Considering the actual experience of the other countries, therefore, and the preceding detailed discussion of the actual effect of gauge, there seems, to the writer, to be little opportunity for a difference of opinion that the 4 ft. 8½-in. gauge is the most suitable for almost all conditions, and that for new lines, even in, or perhaps more particularly in, mountainous districts, and where the amount of business to be expected is not large, the slight difference in cost is generally more than warranted by the possibilities of economy in operation. The question in the Argentine and Brazil, however, is whether the cost of changing the existing narrow-gauge lines is warranted, and, in the former country, whether the change should be to 4 ft. 8½-in. or to 5 ft. 6-in., and, by the adoption of this latter, unify the gauge of all the lines in the country.

The consideration of a change of gauge, if confined to the Argentine alone, is practically narrowed down to the question, whether it is possible to unify the gauges of all the railroads, or whether one gauge shall be eliminated, leaving only two instead of three? The following argument, as far as page 387, therefore, is confined to a consideration of the question as it affects the Argentine, and without reference to the adjoining countries.

The Best Gauge for the Argentine Alone.—The relative advantages of 4 ft. 8½-in. and 5 ft. 6-in. have been discussed at some length, and it is believed that it is shown fairly conclusively that 5 ft. 6-in. does not now (with the imposed restrictions as to loading gauge in the Argentine) offer any advantages, from a purely transportation standpoint, over 4 ft. 8½-in., nor would it be likely to, even if there were a possibility of taking full advantage of the extra width of gauge by designing rolling stock with the same proportionate overhang. It seems important to emphasize this latter statement, as it is generally supposed that the 5 ft. 6-in. gauge could be better utilized if full advantage were taken of the extra width, but this is not the case,

or, at least, only in a minor degree, and, if the 5 ft. 6-in. gauge is adopted, it will not be for any advantage it has over 4 ft. 8½-in., but solely in order to unify the gauges of all the lines in the Argentine.

It is a safe assertion that it is impracticable to work more than two different gauges, inasmuch as all lines must have access to the ocean ports, and although it is practical to work two gauges, by having a third rail, on the port railways, it is not practical to work three, and it is impractical to reserve a special section of a port for a special gauge.

The broad-gauge is undoubtedly there to stay, as it has the greatest mileage, the companies working it are by far the strongest financially in the country, and, to a large extent, are all controlled by mutually friendly interests which probably would not even consider a proposition to narrow the gauge.

The meter-gauge has the next largest mileage, nearly four times that of the medium-gauge, and it would be far more costly to convert the meter-gauge lines to medium- than *vice versa*. It seems, therefore, as if, between these two—the meter- and the medium-gauge—the medium might have to go. This would be particularly unfortunate, as it would be undoubtedly a long step backward, but there are many indications that a sentiment is rapidly growing against the existence of the three gauges.

The Government lines, all meter-gauge (except those in Patagonia which are 5 ft. 6-in.), have been built with the primary object of bringing the outlying sections of the country and the capitals of the Northern Provinces in touch with the National Capital, for both political and military reasons. They reach every provincial capital, except Entre Rios, Corrientes, Mendoza, and Buenos Aires, and are connected with the latter, the National Capital, by two separate lines of meter-gauge, the Central Cordoba and Compañia General de Buenos Aires, so that every capital (except Mendoza) and practically the whole of the northern end of the Chilian frontier and the frontier of Bolivia can be reached by through trains from the capital without change of coaches.

The frontiers of Paraguay, Brazil, and Uruguay, where they join the Argentine, can only be reached by the medium-gauge, and, though this gives access to them from the National Capital itself, if it becomes necessary to send troops to any of these frontiers, all, except those from the City of Buenos Aires and vicinity, would have to be transferred from cars of a different gauge. This, though it may appear to

be a far-fetched argument for changing the gauge of the Entre Rios and North East Argentine Railways to narrow-gauge, at least partly explains the growing sentiment that the Government should be able to reach these frontiers over lines of the same gauge which now enables it to reach every other part of the country except Bahia Blanca, and access to this latter by a meter-gauge line is only a question of time, if the rest of the meter-gauge lines remain as they are. The experience of Japan (see page 400), however, shows the inferiority of the narrow-gauge for military purposes.

There is also the feeling of the people of these two provinces, Entre Rios and Corrientes, that they are isolated. So long as their railroads stopped at the edge of the rivers, and they were obliged to leave the train and cross the water in some kind of a boat, they accepted that as inevitable. The opening up of the Entre Rios car-ferry route to Buenos Aires, however, has completely changed the aspect of the province, and has shown its inhabitants the great advantage of through railway communication, so that they will not be satisfied until they have the same kind of through communications with the great northern section of the country, Tucuman, Bolivia, and the Chaco, by car-ferry transfers between Parana and Santa Fé and between Corrientes and Resistencia.

From Santa Fé there are narrow-gauge lines reaching practically the whole country except the Cities of Bahia Blanca and Mendoza. It is extremely natural, therefore, that it should be proposed, as it is quite seriously, to cover the whole of the Provinces of Entre Rios, Corrientes, and Missiones with a network of narrow-gauge lines connecting at Parana and Santa Fé with the narrow-gauge net on the other side, or to have the Government take over the existing line and change the gauge to meter.

It seems to the writer, however, that these proposals are made without an adequate realization of what an efficient transportation machine is, and what it means to the future development of the country. The need of fairly rapid transportation for passengers, the character of the freight traffic, etc., are indicated in Appendix B.

The principal defect of the narrow-gauge, which has been already pointed out, is its much lesser stability and greater stresses, both to rolling stock and track, caused by the unevenness of the latter, combined with the much greater height in proportion to the width of support of the former. This makes stone ballast—a very expensive article in

the Argentine—an absolute necessity for the narrow-gauge, whereas the broad- and medium-gauge, in many cases, can handle the traffic fairly comfortably without it.

In considering the merits of the two gauges, there seems to be little need, at this late day, to prove that the narrow-gauge, as a transportation machine, that is, regarded solely from the standpoint of a machine for the safe, comfortable, rapid, and economical transportation of passengers and freight, is inferior to medium or broad. Even the most ardent advocates of narrow-gauge admit the inferiority, but claim that this is compensated by the lower first cost, and also, sometimes, that narrow-gauge lines are cheaper to operate.

The statement that narrow-gauge lines are cheaper to build is fallacious, but still persists in some quarters, because many narrow-gauge lines actually are built more cheaply than some medium- or broad-gauge lines. This, however, is not because they are narrow-gauge—at least not to any great extent, as has already been shown in detail—but because they are built of cheaper materials. They have lighter or lesser capacity. For the majority of the lines to be built in the Argentine, which will cost mostly less than \$25 000 per km., the costs would be, approximately, for the same type of construction and including all interest and overhead charges:

Narrow-gauge, per kilometer	......	\$25 000, gold.
Medium-gauge, " "	....	27 500 "
Broad-gauge, " "	....	27 750 "

There will be many hundreds of kilometers costing not more than \$15 000, on which the difference would be even less.

The mistake most often made in discussing this question is in confusing the terms "Narrow-gauge railway" and "Light railway", though, of course, a "Light railway" may be of any gauge.

In some of the discussions on this phase of the subject, it has been claimed that if standard- or broad-gauge roads or branches were built with light rails, bridges, etc., it would not be possible to use the same rolling stock on them that is used on the main line, or lines of heavier construction, but this can hardly be supported. It must not be forgotten that the force of impact due to speed is a most important factor, governing the necessary strength of the track and other structures of a railroad, and though one would not expect to run the heaviest type of

locomotives over a light line, it is perfectly feasible to carry any rolling stock, except the heaviest locomotives, at low speeds on rails as light as 40 lb., and there is no economy in building bridges for any railway traffic that will not carry freight cars of 50 tons capacity running at speeds of 20 miles per hour, which is quite fast enough for branches of light traffic.

In regard to cost of operation, there is room for difference of opinion, as it seems to be impossible to offer proof of the actual effect of gauge on this. The matter has been discussed in some detail, and there seems to be no reasonable doubt that, assuming equal conditions, the cost would be less on the broad-gauge than on the narrow-; but, just how much, it would be difficult to say. Certain of the figures seem to show that the saving made possible by the use of larger rolling stock might alone be more than enough to compensate for the cost of conversion.

The problem, therefore, so far as only the Argentine is concerned, seems to be narrowed down to the question of either the unification of the gauge by conversion of all lines to 5 ft. 6-in., or of changing all the narrow-gauge lines to medium, or all the medium to narrow, and in any case the scheme adopted must embrace all the lines of the gauge to be changed. So far as having an efficient transportation machine is concerned, the problem would be solved by the conversion to the medium-gauge, but if this is to be considered seriously, the question naturally arises would it not be throwing away a great opportunity to go farther and convert all to 5 ft. 6-in., and so unify the gauge of all the lines in the Argentine (except certain of the mountain lines of light traffic which could remain as narrow-gauge feeders, at least for some time)? The additional cost of conversion to the broad-gauge instead of the medium would not be so very great, in comparison with the whole cost in a scheme so large as this, and it would not be unreasonable to assume that, to obtain a result of such far-reaching benefit to the country, the Government of the Argentine, like that of Australia, would assume a large share of the burden.

Although there can be little room to question the statement that, for new lines in a country where construction is as light as it is in the Argentine, the additional extra cost for the wider gauge is so small as not to be worth considering, when compared with its great advantages over the narrow, the important question is the justification,

from a financial standpoint, of spending the amount necessary for the conversion of the existing lines, which, up to the present at least, have not shown such great deficiency in handling their business as would seem to warrant the expenditure of this sum in improvements.

It must not be forgotten, however, that, as pointed out on page 365 and in Appendix B, the character of the narrow-gauge lines is just now undergoing a radical change, from that of more or less small local lines, doing a small local business without competition, to that of a large trunk-line system, many of the lines of which are or will be competitive either with other broad-gauge lines or with river transportation, and they must either develop into an efficient transportation machine, capable of handling such business as may be offered at rates and in a manner which will compare favorably with those elsewhere, or be eventually crowded to the wall by transportation systems which can and will do this.

The underframes and draft rigging of the larger part of the rolling stock of the narrow-gauge lines is so light as to prevent its use in trains of any weight or length. The strengthening of these, or the provision of adequate strength in new equipment, will make the dead weight relatively large in proportion to the paying load, and in every way the narrow-gauge is forever limited in the matter of speed, comfort, stability, and economy of operation when compared with the standard-gauge.

If no great development or extension of the existing system of railroads were to be expected, it is doubtful if the expenditure necessary for conversion would be justified, but if the future development of the country is to be on the scale which a knowledge of it and a study of the general condition of the world's food market seem to indicate, it is believed that it can be.

The possibilities of double-tracking the existing narrow-gauge lines instead of widening the gauge, must, of course, be considered, especially as the cost of this on the lighter lines would be little, if any, more than the cost of conversion, and double-tracking could be carried out as required and without disturbing traffic in any way. It is doubtful, however, if a double-track narrow-gauge line has much greater efficiency or even capacity than an efficiently managed single-track line of standard-gauge (12 000 000 tons per mile of line per annum, or a total of 650 000 000 tons per annum, are handled on the Bald Eagle Branch, a single-track line of the Pennsylvania Railroad, 54 miles in length),

and then the cost of maintenance of double track is practically double that of single track. (See also page 341.) The cost of maintenance per ton-mile for any volume of traffic which might reasonably be expected in any part of the Argentine, except in the vicinity of the National Capital, would probably be almost, if not quite, twice as much for a double-track narrow-gauge as for a single-track wide-gauge, and the possibly somewhat greater ease of handling trains would be more than offset by the other operating economies possible on the wider gauge. In mountainous or even rough country, of course, the cost of double-tracking would far exceed the cost of changing the gauge, and it has been shown that the narrow-gauge offers very little advantage in decreased original first cost in rough country.

Looking at the question from the broad economic standpoint of the future development of the Argentine, it seems to the writer that the policy of continuing the construction of narrow-gauge lines is wrong, especially as it is believed that it can be shown that the additional cost of an efficient transportation machine, which the narrow-gauge is not, can be fully justified from the financial point of view, as well as from the point of view of its absolute necessity for the proper economic development of the country.

Cost of Unification for Argentine Railways Alone.—In calculating the cost, one must consider, not only the cost of changing the existing lines, but looking farther ahead, estimate the additional cost of the completed system as broad-gauge, over and above what it would cost to complete it as narrow-gauge. That is to say, supposing there are 10 000 km. of meter-gauge to be changed and 10 000 km. more to be built; the actual cost of changing might be \$8 000 per km., the cost of building as 4 ft. 8½-in., possibly \$1 000 or \$2 000 per km. more than if built as meter-gauge, so that the added cost per kilometer for the whole of the completed system would be about \$5 000 per km.

Taking the country only as far north as 22° South Latitude (the southern border of Bolivia), there is, north of the line of the existing railway from Sante Fé to Tucuman, a region comprising some 100 000 sq. miles (260 000 sq. km.) in the Argentine alone, and say, roughly, 25 000 sq. miles (65 000 sq. km.) more in Paraguay west of the Paraguay River at present served by only about 1 250 km. of railway. In other parts of the Argentine the length of railways in comparison with the area (for 1909) is approximately as shown in Table 23.

TABLE 23.

	Per 100 sq. miles.
Federal District	152.0 km. = 94.0 miles.
Province of Buenos Aires	7.2 " = 4.5 "
" " Cordoba	5.0 " = 3.1 "
" " Santa Fé	8.0 " = 5.0 "
" " Entre Rios	3.1 " = 2.4 "
Similar units for other areas are:*	
United States	14.1 " = 8.75 "
France	27.8 " = 17.22 "
Prussian-Hesse	40.0 " = 24.08 "
Great Britain	52.6 " = 32.65 "

* *Bulletin*, Bureau of Railway Economics.

It seems reasonable to assume that to develop this northern region there should be at least 5 km. of railway per 100 sq. miles, or a total of 6 250 km., of which there is now in existence some 1 250 km., leaving 5 000 km. yet to be built.

There is in existence at the present time in the whole country about 9 000 km. of narrow-gauge railway, of which 1 000 km. are strictly mountain lines of light traffic which need not be converted for some little time. In the next 20 years there will be required, say 4 000 km. of extensions to the existing lines, outside of this northern zone. So that, taken altogether, mountain lines and all, there will be 9 000 km. of the narrow-gauge to be converted and, say, 9 000 km. of new lines to be built.

To complete the unification of the gauge to 5 ft. 6 in., the standard-gauge lines must be included, of which there will be about 2 500 km. to be converted and about as much more new line required for extensions. The total additional cost of the final broad-gauge system, as compared with narrow-gauge, would then be:

Conversion of 9 000 km. of meter-gauge to broad, at $10 000 per km.	$90 000 000
Extra cost of building, say 9 000 km. of broad-gauge instead of meter, at $2 500	22 500 000
Cost of changing 2 500 km. of medium-gauge to broad, at $6 500 per km.	16 250 000
Extra cost of building 2 500 km. of broad-gauge instead of medium, at $1 500	375 000
	$129 125 000

That is looking ahead, say, 20 years, and not considering the existing broad-gauge lines. The average cost per kilometer of line then in existence as broad-gauge would have cost about $5 000 per km. more than if the existing narrow- and medium-gauges are perpetuated.

The interest on this, at 6%, is approximately $300 per km. per annum, so that, taking into consideration all the figures given in the previous discussions, and the several other undeniable advantages pointed out, it seems entirely reasonable to expect that, even with only a fair amount of business, the earning capacity of the broad- over the narrow-gauge would justify this expenditure, and, in addition to this, it is believed that there are many other reasons, particularly the narrow margin of safety on the narrow-gauge, the necessity of keeping the track in almost perfect condition in order to insure the possibility of even reasonable speeds and a fair amount of comfort, which warrant the expenditure.

The total expenditure indicated in the foregoing makes full allowance for the losses, if any, which might be due to the change of rolling stock. If, as has been proposed, the change was spread over, say, 10 or 12 years, quite a large proportion of the existing rolling stock would be used up during that period, and at the end of that time, in all probability, there would be direct connection with the narrow-gauge lines of the West Coast and the remainder of the rolling stock could be disposed of to roads in those countries, thus greatly minimizing this item; also, the difference in the value of the old rolling stock and the new would be much more than counterbalanced by the economies possible in the operation of the new rolling stock, all of standard types and of similar classes, and all selected in the light of the present knowledge of the requirements of the conditions to be met and the wide range of types now available to meet them.

The Best Gauge for South America.—The foregoing argument is based entirely on the point of view of the Argentine alone, without considering any of the adjoining countries or the necessities of international communications. If the whole of that part of South America referred to, that is, the part shown on the map, Plate IX, is considered, however, the solution of the question as to which of the two wider gauges to adopt, 5 ft. 6-in. or 4 ft. 8½-in., is not quite so apparent.

The adoption of the 5 ft. 6-in. gauge in the Argentine leaves Paraguay and Uruguay with the medium (4 ft. 8½-in.), and it can hardly

be expected that either of these countries, more especially the latter, with some 2 290 km. of line, would consider a change from which it could expect no possible benefit. Brazil, because of the existence of a small quantity of 5 ft. 3-in. gauge, might possibly be inclined to favor the 5 ft. 6-in., but there is not enough of it really to influence the question either way. The 5 ft. 3-in. is neither one thing nor the other, and it would be practically as easy to change it to 4 ft. 8½-in. as to 5 ft. 6-in.; therefore, it seems far more likely that, if the change could be brought about, Brazil would prefer to change to 4 ft. 8½-in. as a standard, thus conforming with Uruguay and Paraguay and the existing lines in Entre Rios and Corrientes.

For the 19 000 km. of narrow-gauge lines now existing in Brazil it would cost about $40 000 000 or $50 000 000 less to convert them to standard-gauge than to 5 ft. 6-in. Looking toward the north, also, the lines of Peru are largely standard-gauge, even where they cross the highest passes of the Andes, and this really is the only logical gauge to adopt if an agreement could be effected between the different countries.

The conversion of the meter-gauge lines of the Argentine to standard 4 ft. 8½-in., and the adoption of that gauge for all new lines north of Santa Fé and Cordoba, and in the rest of the country, except for the legitimate extensions of the existing broad-gauge lines, would be a much less costly proceeding than that estimated above for their conversion to 5 ft. 6-in. It would provide an efficient transportation machine reaching from Buenos Aires to all parts of the country, and in harmony with the network already existing in Uruguay, Entre Rios, Corrientes, and Paraguay, and it might not be unreasonable to expect that, with this in view, Brazil would join in the movement, as there can be no question that such large areas as those under consideration cannot be efficiently developed by a transportation machine inferior to that which has achieved such wonderfully successful economic results in the United States, with which country this part of the world is most nearly comparable.

Making the estimate for the Argentine alone, on the basis of a change to 4 ft. 8½-in. instead of 5 ft. 6-in., and leaving out the existing 5 ft. 6-in. system, the cost of the completed system of standard-gauge roads would be as shown in Table 24.

TABLE 24.—COST, IN GOLD DOLLARS.

Conversion of 9 000 km. of meter-gauge to 4 ft. 8½-in. at, say, $8 500 per km.	$76 500 000
Additional cost of building, say, 9 000 km. of 4 ft. 8½-in. instead of meter, at $2 000 per km.	18 000 000
Extra cost of building 2 500 km. of extensions in Entre Rios, Corrientes and Missiones	nil.
Cost of changing existing 2 500 km. in Entre Rios, Corrientes and Buenos Aires	nil.
Total	$94 500 000

That is, the standard-gauge system in existence at the end of 20 years would have cost about $4 000 per km. more than if the meter-gauge had been perpetuated. The interest on this at 6% would be about $240 per km. per annum, which it is believed could easily be saved by economy of operation, to say nothing of the necessity of having an adequate transportation machine.

Next, take the area south of 22° South Latitude and between the Atlantic Coast and a line drawn from Buenos Aires along the river to Santa Fé and passing through Tucuman and Salta to Sucre, but omitting the section north of São Paulo and Rio, already fairly well served by railways. There are in this area about 1 000 000 sq. miles, served by only about 12 200 km. of railroad or 1.22 km. for every 100 sq. miles of territory.

Of this area, the Province of Entre Rios is, perhaps, the best served at present, with about 3.9 km. of railway per 100 sq. miles, and it is seriously proposed to double this mileage within the next few years.

The railway mileage of the Argentine has increased three-fold in the 20-year period, 1890 to 1910, and it seems not unreasonable to expect the same rate of increase during the next 20 years in the area under consideration. It is true that the climatic conditions are not quite so favorable, but, on the other hand, the era of great expansion in North America, through the development of the country by new lines of railway, is now over, the world demand for pastoral and agricultural products is increasing, even the United States itself is just entering into the class of consumers, so that the development of this area seems inevitable, and will only be held back, if at all, by the lack of capital.

Assuming that there will be 25 000 km. built in this territory during the next 20 years, which will give a density of 3.7 km. per 100 sq. miles,

of this, probably, 5 000 km. will be of standard-gauge, in any event. Then the cost of a standard-gauge system throughout the whole area, including the cost of conversion of all the existing lines except those of broad-gauge of the Argentine, would be as shown in Table 25.

TABLE 25.

Cost of conversion of 28 000 km. of meter-gauge to standard, at $9 000	$252 000 000
Additional cost of building 20 000 km. of line, 4 ft. 8½-in. instead of meter, at, say, $2 500	50 000 000
Cost of conversion of 500 km. of 5 ft. 3-in. to 4 ft. 8½-in., at $1 000	500 000
4 800 km. in Uruguay, Entre Rios, Corrientes, and Paraguay not requiring change	nil.
5 000 km. of new line in Uruguay, etc., etc.	nil.
Total additional cost	$302 500 000

It will be noted that somewhat larger allowances for conversion, etc., are made for this whole system than for the Argentine alone.

The standard-gauge system in existence over this whole area at the end of 20 years, therefore, would cost about $5 000 per km. more than if the meter-gauge were perpetuated, this involving an annual interest charge of, say, $300 per km. The greater cost by including the lines of Brazil is due to the much greater present development of the meter-gauge system there, and also by additional allowances to cover the additional costs, as a great deal of the topography is much more accidented in Brazil than in the Argentine. On the other hand, no account has been taken of the vast areas in Brazil north of Latitude 20°, in much of which, especially in Eastern Brazil, there is likely to be considerable development by new lines, which, if built in the first place as standard-gauge, would reduce the average cost per kilometer of the whole system.

It seems to be not unreasonable to assume that, as many of the narrow-gauge lines will require stone ballast at a cost of from $1 500 to $2 500 per km., where the wider gauge could be operated just as well without, the completed system should be given some credit on this account, which might easily be as much as $1 000 per km. over the whole system, thus reducing the interest charge by $60 per km.

There is also the question of double-tracking many of the narrow-gauge lines, which would not be necessary if they were medium- or broad-gauge, and this would offset quite a little of the cost of conversion, though it is somewhat difficult to say just how much. Suppose, how-

ever, that 15% of the systems as narrow-gauge would have to be double-tracked and half of this, say 4 500 km., could be avoided by the change of gauge, there would be a saving of about $100 000 000 on this item alone, or one-third of the estimated additional cost of the whole system as standard-gauge.

It has been shown that, on the basis of the most conservative estimates, the saving in cost of operation made possible in only the one item of larger rolling stock, might easily be more than $300 per km. per annum, and as it is considered necessary even now to use 70- and 80-lb. rails on the narrow-gauge, this is not offset by the necessity of increasing the strength of the track to take care of the heavier rolling stock, as this weight of rail is ample to carry the ordinary standard-gauge equipment of fairly heavy types, on which these estimates are based.

It seems almost superfluous to point out the advantages of unification of the gauge. Besides the inconvenience of transfer and the actual cost of moving freight from one car to another, which has been variously estimated at from 5 to 10 cents per ton in India,* there is the actual damage to many classes of freight, such as fruit, etc. The transhipment of refrigerated products is impractical. Two cars are standing idle while any transhipment is being made, there are the unavoidable delays in yards waiting for the necessary cars of both gauges to be provided and shifted into position, and, above all, the loss in fluidity of rolling stock, which would be especially noticeable in both the Argentine and Brazil, where, for a few months during and after the harvest, there is an exceptional demand for rolling stock which could be used during the other seasons for hauling timber.

In the Argentine this question of transfer from one gauge to another has not become acute up to the present time, as most of the lines of each of the gauges reach terminal points on the water-front, to and from which most of their traffic flows. With the development of the country, the actual interlocking of the lines of the different gauges, and the present tendency to continue this mixing of the territories, this question, however, is bound to assume greater and greater importance, and the necessary transfers will be the cause of more and more inconvenience and delay.

* *Minutes of Proceedings*, Inst. C. E., "The Railway Gauges of India," by Sir Frederick Robert Upcott, Vol. CLXIV, p. 196.

It is true that the foregoing figures are based largely on assumptions, and may be varied widely in many individual cases, but it is believed by the writer that on the whole they represent a fair average, and at least tend to indicate that the adoption of 4 ft. 8½-in. as a standard, and the conversion of the existing narrow-gauge lines, can be justified on economic grounds and approximately so from the financial viewpoint.

It is hardly to be expected that these figures are definite enough to justify private capital in entering on an enterprise of such magnitude, but, taking into consideration the fact, which it is believed has been demonstrated, that the narrow-gauge is absolutely inadequate for the development of such large areas as those under consideration in the Argentine, Eastern Bolivia, Brazil, Paraguay, and Uruguay, to say nothing of the rest of South and Central America, it may not be unreasonable to expect such a measure of assistance from the Governments involved, which would be justified by the benefits they would derive from a system of railways of uniform gauge covering the whole area.

This argument, it will be seen, is based largely on the supposition of the continued and continuous development of these countries, which seems assured, provided the past policy of encouragement and appreciation of private capital is continued and the political situation remains as stable as it now seems to be. Without foreign capital, of course, this development will be tremendously retarded, to the benefit of competing countries like South Africa, Australia, and New Zealand.

It has been, and still is, the policy of the Governments of both Brazil and Argentine to build as State enterprises lines of railway to connect up or push the development of certain sections not sufficiently attractive at the time in themselves to private enterprise. Of course, the extent to which this can be carried out is more or less limited, as in all these newer countries there is a demand for more capital than is available; therefore, the Government lines, after a while, are sold or leased, and the capital thus acquired is used for further development elsewhere. It seems to the writer that, taking everything into consideration, such lines, if built to standard-gauge, would cost little if any more than have the meter-gauge lines, and there is no question at all but that they would be a far more valuable asset because of their undoubted greater economy in operation and their better salability

because of the wider market for any standard article than for one of inferior quality.

The 5 ft. 6-in. lines of the Argentine probably will not consider the change for the present, but, as they are fairly well confined to a limited area, it will do no harm to leave them as they are. They are efficient, and to attempt to include them in any scheme for unification of gauge would be to create a formidable opposition which it would be difficult to overcome. The adoption of 4 ft. 8½ in. as a standard for new lines, and the gradual change of the meter- and other narrow-gauge lines to the standard, however, should be established as the policy, if the requirements of the future are to be met.

It must be borne in mind that, as the distance from the seaboard increases, the necessity for cheap transportation becomes more and more important, and as this section of the world must depend for its future prosperity on its ability to compete successfully in the food markets of the world, cheap efficient transportation is a *sine qua non*. The United States has achieved results in this line which are far ahead of those obtained elsewhere, almost wholly by reason of the increase in train loading. It is quite certain that no narrow-gauge line can attain anywhere near the economic results obtained in the United States, and nothing which falls much short of this will solve the problem for the area under discussion, which only needs capital and adequate railroad transportation to show the most phenomenal development in the history of the world. This development, however, will be tremendously handicapped with anything less than the efficient, economical transport which can be furnished by standard-gauge railways.

In closing, the writer wishes to express his thanks to Mr. Percival Farquhar, President of the Argentine Railway Company, for permission to use much of the information contained in a report made to that company on this subject by the writer, to Mr. L. E. Young, of the Middletown Car Company, for data in regard to rolling stock, and to Mr. C. M. Muchnic, of the American Locomotive Company, for data in regard to locomotives and other information. It is believed that proper credit has been given in the text for quotations and assistance from engineering periodicals and other sources, but inasmuch as a fairly wide search of the literature of the subject has been made

it is possible that some omissions of credit may have been made, if so, it is needless to say it has been unintentional.

Generally speaking, it is believed that the statistics given in the paper are substantially correct, and in any event such differences as there are do not affect the general argument, so far as the gauge question is concerned. As with all figures relating to railroads, there are often differences due to different methods of calculation or recording, and this is perhaps especially so in regard to South America. The writer, therefore, would esteem it a favor if those who may have more intimate knowledge of any facts will not hesitate to state them with a view to making the record as complete and accurate to date as may be.

APPENDIX A.

History, Experiences of Other Countries, Etc.

The following notes give briefly some account of the unification of the gauge of the railways in the United States and Mexico, the experience with lines of different gauges in India, Australia, etc., and Table 26 shows the length of lines of various gauges now in operation in the different countries of the world.*

TABLE 26.—Length of Railways of Various Gauges in Operation Throughout the World.

All Distances in Kilometers.

	Broad, more than 5 ft.	Medium, 4 ft. to 4 ft. 11 in.	Narrow, 3 ft. to 3 ft. 11 in.	Industrial, less than 3 ft.
Great Britain	4 959	40 017	909	104
Europe (Continent)	71 142	206 912	23 517	8 264
North America	....	447 721	12 394	....
Central America and West Indies	153	3 382	2 596	141
South America	18 852	6 663	37 115	3 094
Asia	43 781	11 650	41 299	2 618
Australasia	6 415	6 179	19 257	226
Africa	....	5 275	27 843	4 089
Totals	145 302	727 799	164 930	18 486

**Note.—The medium-gauge is nearly all 4 ft. 8½-in.
The narrow-gauge is nearly all meter or 3 ft. 6-in.**

United States.—The following, prepared by Mr. George L. Fowler, of Messrs. Hildreth and Company, gives a brief statement of the history of the gauge discussion and its result in the United States.

"The narrow-gauge mania which possessed the railroad builders of the United States in the early Seventies was born of the bargain hunting desire to get something far below its real value. It was at the time very difficult if not impossible to raise money for the construction of new railroads of the standard-gauge. It was just after the civil war, and hard times were upon the country, yet in spite of this fact, the people over the whole land were clamoring for a more rapid development of the resources than was taking place, and this clamor was especially loud in its demand for an increase of railroad construction, but capital could not be obtained.

"It then occurred, to those who were most vitally interested in these constructions, that if cheaper roads could be built, the money for their construction might be obtained. They immediately, therefore, jumped to the conclusion that, because a gauge of 36 in. is only about three-fifths of 56 in., a road of that gauge could be built for a correspondingly lower price, and straightaway positive statements were made to that

***Compiled from "Universal Directory of Railway Officials," 1912.**

effect, which were honestly accepted by a large number of men. It was assumed that, because the gauge was to be narrowed, all costs would be cut down in a corresponding degree. It was argued that because of the narrow-gauge, the cars would be made lighter than those which it had been found advisable to use on the broad-gauge, forgetful of the fact that equipment of the same, or even lesser, weight could and had been built and used on the standard-gauge lines, and had been found unsatisfactory.

"The whole case rested, apparently, so far as the United States was concerned, on the inability to raise money for ordinary railroad construction, and the narrow-gauge seemed to promise greater dividends on a given investment. It was characteristic of the whole discussion that the advocates of the narrow-gauge insisted throughout that their construction was the cheaper, and could not or would not see their opponents' side of the case, which was, that the narrow-gauge could not be built materially, if any, cheaper if the same facilities and capacity for the transportation of freight and passengers were to be afforded. They failed to see or acknowledge that a light locomotive and car could be built for a broad-gauge line.

"The physical disadvantages of the narrow-gauge, such as cramped storage and seating capacity, the greater instability of the rolling stock, and, above all, the chief drawback of the impossibility of interchanging cars with the standard lines, with the consequent extra cost and delays in transportation, were pointed out again and again, but in spite of all this the narrow-gauge construction went on for several years, and, until the money market eased, made a great headway.

"Meanwhile, the disadvantages of different gauges were making themselves manifest, and it was also being realized, with great rapidity, that heavy loads meant lesser costs than when light ones were hauled, so that between 1873 and 1885 there was a very rapid increase in the carrying capacity of the standard-gauge cars, rising as it did from 10 to 30 tons. With this rise the dead weight of freight cars dropped from a ratio of 1 lb. of dead weight to 1 lb. of paying load to ½ lb. of dead weight to 1 lb. of paying load.

"When the necessity of car interchange drove the broad-gauge lines to change to the standard (4 ft. 8½-in.) gauge, the narrow-gauge lines were left in a state of greater isolation than they had been at the outset of the movement.

"Finally, the recuperation after the panic of the Seventies made money more easily procurable for railroad construction, and the narrow-gauge roads, which had been built and equipped in the lightest and cheapest possible manner, found themselves doubly handicapped by their lack of facilities to do business, and by their isolation.

"The result of all of it was that, the furore past and the money market relieved, the narrow-gauge mania died a natural death, if a death from lack of substance can be called natural. The result then, of the forty years of practical experience that has elapsed since the battle of the gauges was precipitated in the early Seventies, has been to quite discredit the narrow-gauge system. It has been proven beyond all peradventure that the narrow-gauge railway cannot be operated so as to keep its ton-mile costs down to the figure obtained on the standard-gauge roads, and this has been the cause of its demise.

"Cars and locomotives of present capacities are impossible on a 3-ft. or 3 ft. 6-in. gauge, and time has shown that the contentions of the early opponents of the system were correct and that the narrow-gauge railroad cannot compete with the broad-gauge in capacity, facilities offered, or in cost of operation, when this cost is put upon the ton-mile basis."

The Denver and Rio Grande was the most important of the narrow-gauge lines changed to standard, and is interesting from the fact that it is a mountain line, of steep grades, very sharp curvature, and heavy work, crossing as it does the mountainous region of Colorado from Denver to Salt Lake City.

The change was started in 1886, during which year almost half the mileage (1 200 miles) was changed, the rest being done as the finances of the road permitted, and less than 6 months ago (September, 1912) it was decided to complete the last section, over Marshall Pass, a distance of 236 miles, the cost of which work, with the improvements in the line, is estimated to be about $2 000 000, or about $8 500 per mile.

It is of interest to note that on this line, which is now a link in a transcontinental route, and reaches an elevation of more than 10 000 ft., there are grades of 3.8% and curves of 150 ft. radius operated by heavy locomotives, and on which passenger trains of standard Pullman equipment are operated.

Many other lines in the United States were changed, but the largest mileage was in changing from 4 ft. 10-in., 4 ft. 11-in., and 5 ft. 0-in. to the standard 4 ft. 8½-in., this involving some 25 000 miles of various lines, mostly in the South. The total mileage of narrow-gauge changed to standard was probably about 5 000.

Mexico.—The principal railways in Mexico were built to standard gauge, 4 ft. 8½-in.; the gauge of the Mexican National, however—the original trunk line from the frontier of the United States to the City of Mexico, 1 200 miles in length—was made 3 ft. 0-in. Notwithstanding that it had a much shorter direct line to the United States, it was unable to compete with the Mexican Central and the International. In his Annual Report in 1889 the President of the Company said:

"Having been brought by experience to a realization of the inadequacy of a narrow-gauge road to develop a thoroughly satisfactory transportation service for a large volume of business * * * induced your management to take up some time ago the study of the practicability and desirability of changing the track to standard-gauge."

The line was changed to standard gauge between 1902 and 1904.

India.—The gauge of the Indian Railways has been the subject of almost endless discussion. Although nearly all who are familiar with the subject agree that a difference in the gauge of continental railways, or in countries as large as India, is unfortunate, all the discussion and the various reports of eminent engineers and transporta-

tion experts have led nowhere, and lines of both gauges are continuing to be built, apparently, however, only because the problem of conversion seems to be too big to handle. A few quotations from some of the discussions will show the attitude of some of the most prominent British engineers who have studied the subject. The matter was brought up officially for discussion by the Institution of Civil Engineers in 1906, the paper and discussion at that time occupying 135 pages of the *Minutes of Proceedings,* and, previous to that, at intervals as far back as 1873,* the following, at that time far-sighted, opinion was expressed:

"While the 3½-ft. gauge might answer for the carriage of heavy minerals in special districts, the general commerce of every populous country mainly consists of articles of low or medium specific gravity, such as food, clothing, fuel, etc., averaging about 80 cu. ft. per ton weight, for which the 5½-ft. gauge is in every respect most suitable as regards costs, stowage, safety, economy, the intricate elements of military defense, and the power of adopting single-track lines of railways for the accommodation of a large amount of traffic."

Mr. Thomas Robertson was appointed in 1903 as Special Commissioner by the British Government to report on the administration and working of Indian Railways, and in his report he pointed out the difficulties of working the two gauges, meter and broad, as follows:

"It will generally be admitted that a break of gauge is a drawback always, and that in certain eventualities it might prove extremely inconvenient. It necessitates a great deal of expense in the transhipment of traffic and a very much larger supply of rolling stock, since the unavoidable delays during the progress of transhipment absorb a large quantity of rolling stock of both gauges.

"The question is so full of difficulty and the evil is so far advanced that it is not easy to advise on it.

"Uniformity of gauge is bound to be demanded some day, and the longer a settlement of the policy to be pursued is deferred the greater will be the difficulty and expense of introducing it."

He discusses the relative merits of the two gauges, and favors the meter-gauge, largely because of the greater width of rolling stock in proportion to the width of the gauge. (This additional width, as is shown elsewhere, is not an unmixed blessing, as it is gained at the expense of safety and stability.) He concludes by advocating the unification of the gauge by the adoption of the medium-gauge, 4 ft. 8½-in.

In 1906 a paper was presented before the Institution of Civil Engineers of Great Britain by Sir Frederick Robert Upcott, Chairman of the Indian Railway Board, who pointed out the facts in regard to the gauge of Indian Railways without much comment. They were stated to be:

* And in *The Railroad Gazette,* November, 1872.

That the internal communications of the country were more important than any considerations of communication with other countries;

That the bulk of imports and exports was by sea;

Large internal trade;

The attempt to confine the narrow-gauge to a particular zone was useless, as these lines must find an outlet to the coast, and this created a great confusion at the ports by reason of difference in gauge;

The trouble and inconvenience of transhipment were considerable, and would by no means be measured by the cost of actual transfer of articles, but involved rolling stock standing idle, lack of fluidity of rolling stock, etc., etc.

Some of the discussions were, briefly, as follows:

Sir Guilford Molesworth had nothing to say against the meter-gauge as a gauge, but he did think the standard-gauge (5 ft. 6-in.) was better suited to the bulky agricultural traffic of India.

Sir George Bruce stated that when the discussion of this question came up in 1873 he estimated that the difference in cost of construction was very large, being based on the misconception that the width of gauge was the element which regulated the cost. It did nothing of the kind. With a coach of a given width, nothing was gained by putting the wheels closer together.

Mr. R. W. Egerton, commenting on a suggestion to change broad-gauge to narrow-, said:

"It would be economically unsound to substitute an inferior means of transportation for a superior one, at the same time increasing the capital cost * * *. The meter gauge, on account of the low limit of speed attainable, was an unsuitable gauge for large trunk-lines."

Mr. F. E. Robertson observed that most discussions on the gauge question had been vitiated by the implicit assumption that all the dimensions were geometrically similar, and, also, by the omission to remember that speed was a commodity which was sometimes worth money.

The difference in running speeds between the two gauges was not really known, for the broad-gauge (in India) was in this respect usually worked in such a leisurely way, that the meter-gauge had no difficulty in keeping fairly close to it.

Sir Frederick Robert Upcott, replying to a statement that high speed was not required in India, stated:

"Mr. * * * was incorrect in his views as to the need of rapid travelling in India. India, * * * was like every other country, and had progressive ideas of speed, comfort and safety, and the Railway Administrations would have to meet these needs for all classes."

Africa.—In connection with this same discussion of the gauge of the Indian Railways, there were the following expressions of opinion in regard to the gauge of the African Railways:

The General Manager of the Cape Railways wrote (1906):

"There has been considerable agitation on the subject of the laying of light railways during the last few years. I see no reason for changing the opinions I expressed in 1891 and 1892 on this subject, and I trust that Parliament will gravely consider the probable effect of the introduction of a narrower gauge than that of our own railways before authorizing such a breach of gauge. If the gauge originally adopted, 4′-8½″, had been continued instead of changing to 3′-6″ the journey from Capetown to Johannesburg would be performed in about half the present time. The effect of such saving of time in the passenger traffic would have been enormous, but if the present gauge is to be still further reduced, what will posterity think of the foresight, or want of foresight, in adopting a standard that limits the speed and carrying capacity of the trains? When the traffic is not expected to be heavy I see no reason why railways should not be constructed with less ballast than a standard line. When the traffic improves, the line could be better ballasted and prepared for quicker speeds and heavier loads."

C. O. Burge. "The line from the Zambesi river through to Cairo had been projected on a *miserable 3′-6″ gauge.* It would be found that the steamship companies, who were progressing faster than the railway projectors, would carry passengers from North to South Africa much more quickly by steamer than the railway could on a line of 3′-6″."

Japan.—Japan, after a most careful examination of the question, has decided to change the gauge of its railways to 4 ft. 8½-in. The 3 ft. 6-in. lines of that country were found to be entirely inadequate for the transport of troops and war material during the Russo-Japanese war, and, after a most careful consideration of the whole question, it has now been decided to make the change, the estimated cost of which is said to be about $150 000 000 for the 5 000 miles of line.

The South Manchurian Railway was originally 5 ft. 0-in. gauge and during the Russo-Japanese war was converted by the Japanese to 3 ft. 6-in., this being chosen because only rolling stock of that gauge was available. After operating it as a narrow-gauge road for 2 years, however, it was changed to 4 ft. 8½-in., corresponding with the gauge of the Korean Railways, and it has been stated that the Japanese have felt that they owed a debt of gratitude to the American engineers who built the first Korean railway and thus established a gauge which was of great service to them during the war, the 3 ft. 6-in. gauge which they were obliged to adopt temporarily for the South Manchurian Railway having been found entirely inadequate.

Australia.—The situation in Australia is described in the following quotation:*

* *Engineering Record*, September 28th, 1912.

"By some unfortunate misunderstanding, each now laying the blame on the other, New South Wales and Victoria started railways in the Fifties on different systems, the first on the standard, or 4 ft. 8½-in., and the latter on the 5 ft. 3-in. gauge, and, notwithstanding the advice of the engineers of the day, persisted, until now we have two great groups of over 3 500 miles each * * *. The slight difference of 6 in. stands much in the way of even the very inadequate remedy of a mixed gauge, owing to the difficulty and expenses connected with the construction and working of the switches and frogs in such a case.

"Subsequently, Queensland and Western Australia, attracted by the expected but exaggerated economies in the construction and working of narrow-gauge lines, which were too hastily assumed to be synonymous with light lines, adopted 3 ft. 6 in., and South Australia, after most of its main lines had been made to correspond with the width [5 ft. 6 in.] of those of its neighbor Victoria, introduced a local diversity by making its extensions and branches on the smaller, 3 ft. 6-in., basis.

"Unlike the United States and Canada who repented comparatively early in this matter, but at a cost of a large amount of money, Australia has done nothing but talk about it, and, owing to the enormous extension of each gauge in recent years, it is quite out of the question now to think of universal conversion, which might cost anything up to $60 000 000."

Finally, after the federation, it was decided to build a transcontinental line to unite the extreme easterly and westerly States of the Commonwealth, and, of course, the question of the most suitable gauge came to the front at once. After long debate and after calling to its aid the most expert advice, both engineering and other, that it was possible to obtain, the 4 ft. 8½-in. was adopted.

There still remained in abeyance the question of the gauges of the existing lines, but it has just recently been announced* that it has been decided to unify the gauges of all the lines, adopting 4 ft. 8½-in. as a standard and, in view of the general benefit of this to the whole country, the greater part of the burden of this change, estimated to cost about $175 000 000, will be borne by the Commonwealth as a whole.

As an example of the misleading statements often made in discussing the relative value of the gauge, the following is worthy of notice because it is so recent, and made by what one would suppose might be a competent authority.

In December, 1912, speaking at the Royal Colonial Institute in London, Sir Thomas Robinson, Agent General of Queensland, in describing the 3 ft. 6-in. railways of that part of Australia, made the statement that "Queensland had constructed 4 266 miles of railways of this gauge at practically half the cost of 3 807 miles of 4 ft. 8½-in. gauge in New South Wales." He omitted, however, to mention the fact that the total net earnings of the latter were more than double

* *Railway Age Gazette*, July 11th, 1913.

those of the former. He also stated that "in the working of the railway the comparatively light weight of the rolling stock gave it a great economic advantage over railways constructed on the wider gauge," yet said later, "Coming to rolling stock, the lighter engines and cars of the early days ran on lighter sections of rails, but the increased weight and power of the engines had necessitated the relaying of the permanent way with far heavier rails, * * * and the use of this heavier rail was steadily and surely extending from one end of the State to the other." He claimed that break of gauge has few disadvantages, yet stated that a new route has been laid out to the border of New South Wales on which "all the stations, tunnels, etc., had been built sufficiently wide so as to run the New South Wales trains (4 ft. 8½ in.) right through".

The consensus of all these opinions, from a wide range of territory and a very large number of prominent men, covering practically the whole of the railway systems in the English-speaking parts of the world, is decidedly against the adoption of lines of different gauges in the same country, and almost entirely in favor of the broad (4 ft. 8½-in. or 5 ft. 6-in.) as compared with the narrow (3-ft. to 3 ft. 6-in.).

APPENDIX B.

General Description of the Topography, Physical Characteristics and Business Conditions of Southern South America; The Development and Present Situation of its Railways.

In the following, the Argentine is referred to perhaps more particularly than any of the other countries, but this only because the writer is more intimately familiar with it, and the data he has available relate principally to the railways of that country. The conditions there, however, are quite generally similar to those elsewhere in this south temperate zone, and may be assumed to be applicable throughout most of this area and to its railways.

The northern limit of this section for convenience may be assumed to be about Lat. 20° S., that is to say, approximately equal to that of the southern coast of Cuba and the City of Mexico; its southern limit will be taken as the City of Bahia Blanca, Lat. 39° S., which is equivalent to that of Baltimore. The latitudes of some of the more important cities may be compared with some of those of the northern hemisphere, as follows:

Rio de Janeiro,	Havana.
Santos and São Paulo,	Key West.
Antofagasta,	Key West.
Asuncion,	Cape Sable, Florida.
Valparaiso and Santiago, Chile,	Atlanta, Ga.
Buenos Aires and Montevideo,	Memphis, Los Angeles.
Bahia Blanca,	Baltimore, Denver.

It should not be inferred from this that there is no development in that part of the Argentine south of Bahia Blanca and Neuquen, and including Patagonia.

The railway development in this section, however, is not large, and is most likely to be dominated, as far as gauge is concerned, by that of the four important lines in the Province of Buenos Aires, which are 5 ft. 6-in.

The principal topographical features are the main range of the Andes and the River Parana, with its tributary the Paraguay, and the River Uruguay. The Andes occupy a comparatively narrow strip along the Pacific Coast, some 150 to 200 miles wide, broadening out at the northern end of Chile and in Bolivia. Between this range and the Parana and Paraguay Rivers is the vast almost level plain reaching from considerably south of Bahia Blanca to well up into Bolivia, which comprises nearly the whole of the Argentine. Uruguay, Paraguay, the two western provinces of the Argentine, Entre Rios and Corrientes, and the territory of Missiones, are considerably rolling to hilly. In Brazil the land rises abruptly from the coast line to elevations of 2 000

ft. and more, and then slopes generally toward the Rivers Uruguay and Paraguay, the intervening country being mostly rolling and in some parts rather broken and accidented.

In the Argentine there are few, if any, rivers of importance between the Parana and the Andes, the drainage from the easterly slopes of the latter, which is considerable, disappearing in the sandy soil before it has traversed the plain for any considerable distance. Toward the north, above the confluence of the Paraguay and Alto Parana, there are two important tributaries, the Bermejo and the Pilcomayo, the latter forming the northern boundary of the Argentine and dividing it from Paraguay and Bolivia. East of the Parana and Paraguay the country is generally well watered and the bridging of the rivers is an important item.

Lest the foregoing should convey the idea that much of the area of the Argentine is arid, it may be well to note further the peculiar geographical formation of this plain. The top covering of soil, which is of the richest character, and similar in appearance to that of the so-called "black waxey" belt of Northern Texas, at the Parana River and for some distance back from it, changes to a more friable and at times almost sandy loam farther back, and is underlaid by the so-called Pampean mud or "Tosca", a hard, indurated clay, almost shale. The water from the mountain finds its way between the top soil layer and the "Tosca" and between the successive layers of this latter, and is found almost everywhere by comparatively shallow borings. Nearly all of this is good or at least fair drinking water, but some is alkaline or slightly saline, and, therefore, bad for locomotive uses.

The railroad development of the Argentine, therefore, has gone ahead under the most favorable conditions, as far as facility of construction is concerned, and there has been practically no excuse at all for the adoption of the narrow-gauge. Some of the Government lines, notably that running from Tucuman north to La Quiaca, on the border of Bolivia, are in a mountainous country, but the proportion of the total is very small, and may be imagined when it is stated that more than 90% of the alignment is tangent.

Perhaps the most important physical characteristics which affect railway construction and operation are the scarcity of stone or gravel and the high cost of fuel. The coal that is used has to be imported from Europe. The scarcity of stone makes the question of ballast important, as the broad- and medium-gauge lines stand up much better with only earth ballast than do the narrow-gauge lines, and in comparing the quality of the track, the experience of the writer with the railways of the Argentine, leads him to think that it is almost fair to say that a fair stone-ballasted narrow-gauge track is no better than a good earth-ballasted wide-gauge.

The Argentine, as is perhaps well known, derives its wealth from

its pastoral and agricultural products. There are no manufactures of importance, as there is an almost entire lack of minerals and fuel, except wood. That part of the Argentine which, thus far, is developed most, as may be seen by the density of the railway lines, is in the Province of Buenos Aires and the southern parts of the Provinces of Cordoba and Santa Fé. This is an almost treeless plain, formerly devoted to little else than cattle raising, but now gradually being converted to agricultural uses, and the raising of the finer grades of cattle and horses, alfalfa replacing to a large extent the original native grasses. Wheat is cultivated mostly in the southern part of the Province of Buenos Aires and in the vicinity of Bahia Blanca, and exported largely from that port. Maize and linseed are the important crops farther north and in Entre Rios. The principal refrigerating and packing plants are on the Rio de la Plata and the River Parana between La Plata and Rosario. Quebracho comes from north of the City of Santa Fé, and nearly all the exports of it are from that port.

With the great valorization of the lands in this section, due to its agricultural development, the cattle industry is being pushed toward the north, and there seems to be little doubt that the world will soon have to look to this section for much of its meat supply, for the production of which there are vast areas of grass lands available in that part of the Argentine north of Santa Fé and reaching up into Bolivia and Paraguay, in many parts of this latter country, in Uruguay, and in Southern Brazil.

Up to the present time, nearly all the structural timber used in the Argentine has been brought from Europe and the United States, but it is now known that there are large forests of pine and cedar in the States of Parana and Sta. Catharina, in Brazil and in the Territory of Missiones. In Paraguay there are forests of important hardwoods which will now be made commercially available, as there has just been opened up through railway communication between Asuncion, through Corrientes and Entre Rios, to Buenos Aires, and a new line is under construction from Villa Rica on the Paraguay Central toward the east which will probably open up important timber industries and reach the forests where much of the yerba maté grows.

At Tucuman, and farther north between Salta and Embarcacion, there are important sugar industries, and the business of raising early fruits and vegetables for the markets of Buenos Aires, Rosario, Montevideo, etc., is already assuming considerable proportions. Mendoza and San Juan are the centers of quite important grape and wine industries, producing annually some 3 000 000 hectoliters (66 000 000 gal.) of wine, in addition to the grapes which are shipped in very large quantities during the season all over the country and even to Europe and the United States.

The great and important product of the so-called Gran Chaco, which

embraces in the Argentine practically all that territory in the triangle with Santa Fé as its apex and the Pilcomayo River as its base, is quebracho. These trees are not unlike scrub oak in appearance and are generally too twisted and crooked to be of use for structural timber. The wood is very hard and durable, and is used for railroad ties, fence posts, and telegraph poles, the trimmings, roots, etc., for firewood, and the smaller branches for charcoal. It is very rich in tannin, and the whole logs and the extract are shipped in large quantities to Germany for tanning purposes. The whole area of the triangle referred to, so far as known—and quite a little of it has not yet been explored—comprises large areas of forests with much quebracho interspersing the open spaces where the rich grasses provide excellent grazing for cattle. There are some parts which are alkaline, some swampy, some waterless, but, on the whole, it is believed that the greater part of it is useful especially as a cattle country and where maize —for feeding on the spot if not for export—linseed, cotton, tobaccco, sugar cane, etc., can be grown. The quebracho forms the bulk of the freight for the Santa Fé Railway and the southerly end of the Government lines, and this is a product which demands rolling stock of large capacity for its transportation. It is of considerable importance, as it provides paying freight for new lines from the beginning, while the country is being developed pastorally and agriculturally.

Buenos Aires is by far the most important city in the Argentine. It is both the National Capital and the commercial center, its population, 1 300 000, is one-fifth of the total of the country; it receives 80% of the imports, and the exports passing through it are 60% of the total.

The principal other ports are Rosario, the second most important city, with a population of about 100 000, La Plata, the capital of the Province of Buenos Aires, Bahia Blanca, and Santa Fé. The imports which come to these places are mostly coal and lumber, but cereals are exported from all of them as well as from quite a number of smaller places along the rivers where ocean-going steamers can lie alongside the bank and load, being served by short spurs of the railroads tapping the districts adjoining. The Port of Buenos Aires is served by a Port Railway of 5 ft. 6-in. gauge operated by the Government, with a strong feeling against admitting any other gauge. The Port of Bahia Blanca has been developed by the railroads, is served by three broad-gauge railways over their own lines, and has had a very rapid development. The Ports at La Plata, Rosario, and Santa Fé, have mixed gauge (5 ft. 6-in. and meter), the first and last operated by the Government, and Rosario by a French Corporation under a concession for its exploitation.

Most of the merchandise is imported through Buenos Aires and distributed from that city throughout the country, the imports passing through this port being about 6 500 000 tons for 1910, and the exports about 2 000 000 tons. A small proportion of the imports, about

25% of the total tonnage, and consisting principally of lumber and coal, comes directly to Rosario, and in much lesser quantities to Bahia Blanca, La Plata, and Santa Fé, with a scattering to some of the smaller ports.

The cereals are exported principally from Rosario and Buenos Aires in about equal quantities, with Bahia Blanca the next in importance, and then a scattering from a considerable number of smaller ports along the Parana and a few on the Uruguay. The cereals flow naturally to the nearest seaport, ocean-going steamers reaching as far as the City of Santa Fé on the River Parana and Concepcion del Uruguay on the Uruguay River.

Cattle (live) are shipped from all over the country (see Fig. 5) for distribution to the packing plants on the river between La Plata and Rosario. The cattle from Corrientes and parts of Entre Rios go to the plants along the Uruguay River which take, principally, the inferior grades for extracts, dried beef, etc. There is also a considerable movement of cattle from one section to another for growing, fattening, etc., and for shipment to the west coast by driving across the mountains in summer.

There is good passenger business on nearly all lines, sleeping and dining cars being in general use.

The conspicuous feature of the freight traffic, of course, is the cereals. There is a heavy demand for cars during the shipping season, February to June, and in this period quite a large proportion of cars are returned empty, this unbalancing of the traffic being noticeable on nearly all the lines.

In Brazil, of course, the most important products are coffee and rubber, the former coming from that section of the country to the north of São Paulo and Rio. This territory, as will be seen by Plate IX, is served by a fairly dense network of railways which, however, are of two different gauges.

The following table shows the quantities of the principal items of export for 1906. The cotton, sugar, cocoa, and tobacco, are mostly from Pernambuco and Bahia to the northeast of Rio; rubber, of course, comes from the Amazon, and the other products from the States to the south of Rio. The quantities are long tons.

Sugar	85 000	tons.
Cotton and cotton seed	35 000	"
Rubber	62 200	"
Cocoa	25 000	"
Tobacco	23 600	"
Coffee	840 000	"
Maté	57 800	"
Flour and bran	31 500	"
Hides	27 500	"


BOLIVIA
PARAGUAY
BRAZIL
URUGUAY
PACIFIC OCEAN
CHILE
ARGENTINA
JUJUY
6/8
SALTA
FORMOSA
5/4
EL CHACO
MISIONES
TUCUMAN
SANTIAGO DEL ESTERO
CATAMARCA
44/51
CORRIENTES
33/8
LA RIOJA
4/3
13/19
SANTA FE
33/110
SAN JUAN
Santa Fe
ENTRE RIOS
CORDOBA
24/24
Rosario
SAN LUIS
12/12
17/84
Buenos Aires
MENDOZA
6/33
21/99
BUENOS AIRES
EL PAMPA
31/43
19/144
NEUQUEN
RIO NEGRO
1/23
CHUBUT
NUMBER OF CATTLE AND SHEEP PER SQUARE KILOMETER IN ARGENTINA IN 1911.
Upper Figures, Cattle 21
Lower Figures, Sheep 99

FIG. 5.

The principal products of the three southerly States of Brazil, namely: Parana, Sta. Catharina, and Rio Grande do Sul, are timber, some agricultural products, and the yerba maté, of which latter thousands of tons are gathered annually and shipped to points all over the southern part of Brazil, Uruguay, and the Argentine. The development of this section will probably be largely along agricultural and pastoral lines, together with the exploitation of the forests of timber in the western sections.

Uruguay is likely to remain an almost entirely pastoral country, with some agriculture, and, as it is served by a fairly efficient system of standard-gauge railways which shows a healthy normal growth, it does not call for much detailed consideration in the present discussion, except perhaps to call attention again to the fact that this section, Uruguay, the Provinces of Entre Rios and Corrientes, and above them Paraguay, forming a connected area as shown by the map, is served exclusively by medium-gauge lines.

Chile, although one of the most important countries in South America, has little influence on the present discussion, inasmuch as it is cut off from the vast areas to the east by the high cordillera of the Andes. Its railway development was originally that of a series of lines from the mountains to the coast, and more or less perpendicular to the latter, although in recent years, under the influence partly of the scheme for a Pan-American railroad and partly for the development of its internal communications by a means independent of the coastwise steamers, it has put under construction the so-called "Longitudinal" railway, traversing the narrow strip along the foot-hills through the length of the country. The existing and proposed international connections of the Chilean Railroads are noted later.

Bolivia has been little developed, except for the exploitation of its mineral resources, and what development there is has been principally in the high mountain plateau to the south of La Paz. This has been reached from the south by the 2 ft. 6-in. line from Antofagasta, which, at its highest point (on a branch) reaches an elevation of 15 809 ft., the highest railway in the world, and from the north by the Southern Peruvian line (4 ft. 8½-in. gauge) from Mollendo and Cuzco. Within the past year the Arica-La Paz line (meter-gauge) has been completed, giving Bolivia an additional outlet to the Pacific, but all three of these lines have very steep gradients, and summit elevations of 15 000 ft. or more, so that they have only a limited capacity. It is expected that eventually there will be a considerable development of the eastern side of Bolivia, on the lower slopes of the mountains, and on the vast well-watered plains between them and the State of Matto Grosso in Brazil, and the outlet from this section of the country must inevitably be toward the Atlantic seaboard, either south through the Argentine, which, if longer, is through much easier country, eastward through Para-

guay and Brazil to Rio de Janeiro and Santos, or northward through the Valley of the Amazon.

All this vast section seems to be entering on an era of development fostered by an ever-increasing tide of emigration from Europe, comparable only to that of the United States, and when it is considered—to take only one example—that the commerce of the City of Buenos Aires is growing at a rate at which it will probably be doubled in less than 10 years (the commerce of New York is doubled in about 20 years) it will be seen that the question of the policy of the future development of the railways is one of great importance. Figs. 6 and 7 show the increase in trade, exports, and imports, and the areas under cultivation for the three most important cereals.

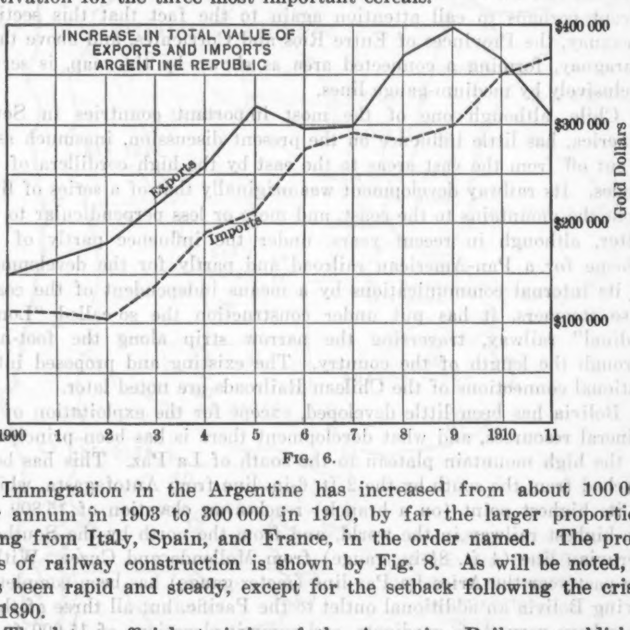


FIG. 6.

Immigration in the Argentine has increased from about 100 000 per annum in 1903 to 300 000 in 1910, by far the larger proportion being from Italy, Spain, and France, in the order named. The progress of railway construction is shown by Fig. 8. As will be noted, it has been rapid and steady, except for the setback following the crisis of 1890.

The latest official statistics of the Argentine Railways published are those for 1909, but Table 27, showing the length in operation, capitalization, earnings, etc., of the various lines for the year ending December 31st, 1912, is approximately correct.

The four broad-gauge lines are by far the most important. They have been built, and are controlled, by British interests. These lines radiate from the City of Buenos Aires, and have been most influential in the development of the country. They are so strongly intrenched

that they are a most important factor to be kept carefully in mind in any consideration of the question of gauge, as it seems entirely improbable that their owners and managers can be shown that there would be any benefit in changing.

The narrow-gauge lines of the Argentine have been principally developed in the section north of Rosario and Cordoba, and, until within comparatively recent times, partook more of the character of small local lines than of a connected system of trunk lines, none of them having reached the National Capital.

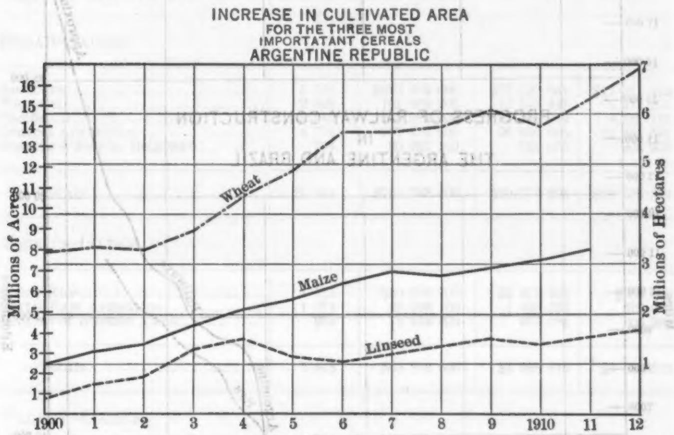


FIG. 7.

In 1908, however, some French capitalists built a narrow-gauge line from Buenos Aires to Rosario and there connected with the lines of the much older French company operating the Province of Santa Fé Railway reaching to Resistencia at the confluence of the Paraguay and Alto Parana, and connecting at Santa Fé with the Government lines through Tucuman to La Quiaca on the Bolivian border.

The Central Cordoba system, which had been formed by uniting some three or four small lines between Rosario, Cordoba, and Tucuman, with headquarters at Cordoba, decided about this same time to build an extension to Buenos Aires from Rosario, and thus connect its system with the National Capital, this line being opened to public service only about a year ago, and the headquarters of the system transferred to Buenos Aires. The Central Cordoba connects with the other part of the Government system, the Argentino del Norte, and also with the Central Norte at Tucuman.

The narrow-gauge lines are thus divided now into three groups: The French lines radiating in three directions from Buenos Aires and

extending north along the Parana up to Resistencia; the Central Cordoba starting from Buenos Aires and passing through Rosario and Cordoba to Tucuman, and the Government lines reaching all the capitals of the northern and Andean provinces and scattered along the foot-hills of the mountains between San Juan and Bolivia, connecting at Cordoba and Santa Fé with the other lines and thus having through communication with Buenos Aires.

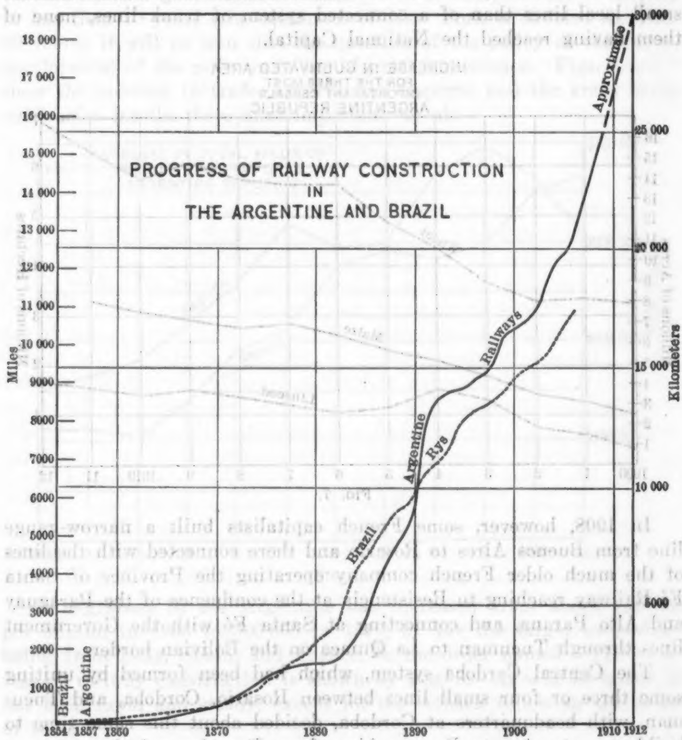


FIG. 8.

These conditions on the narrow-gauge lines must be kept well in mind, as they are just now emerging from a status as more or less isolated lines, doing a small local business, to that of a more or less coherent group of systems of trunk lines connecting the National Capital with the northern section of the country, and reaching nearly all the provincial capitals. As local lines doing a small business, the gauge in itself mattered little, but now, with the greatly increased

business that is coming to them, and the growing necessity of running through express trains at fairly good speeds, they find themselves handicapped by reason of this very increase in business, because they have not the facilities for doing it properly.

TABLE 27.

	Length, in kilometers.	Capitalization.	Earnings.	
			Gross.	Net.
Broad-Gauge.				
Southern	5 608	$220 503 600	$27 127 561	$11 311 439
Western	2 669	101 959 200	12 225 441	5 474 275
Pacific	5 342	216 086 900	24 405 713	9 185 113
Central Argentine	4 751	197 276 600	26 360 005	11 481 579
Rosario & Puerto Belgrano	794	30 957 100	553 575	903 958
Totals	19 164	$766 783 400	$90 672 295	$38 356 364
Medium-Gauge.				
Entre Rios	1 175	$30 391 700	$2 379 390	$908 096
North East Argentine	1 074	29 538 100	1 609 335	622 090
Central of Buenos Aires	269	8 942 800	977 091	305 389
Totals	2 518	$68 872 600	$4 965 816	$1 835 575
Narrow-Gauge.				
Government lines	4 018	$121 872 900	$6 292 069	$359 428
Central Cordoba	1 935	70 525 000	8 220 351	2 253 352
Santa Fé	1 709	42 131 700	5 787 433	2 085 278
Province of Buenos Aires	1 267	39 399 300	2 497 010	527 847
Transandine	185	8 902 800	676 605	125 299
Totals	9 114	$282 831 700	$23 473 468	$5 383 204

It will be noted that the broad-gauge railways have through lines from Buenos Aires to Rosario, Cordoba, Santa Fé, and Tucuman, and, by reason of their long establishment, better track, and more comfortable, because easier-riding, rolling stock, have up to the present monopolized the through passenger business to and from these points. In order, therefore, to get their share of this business, the narrow-gauge lines will be forced to make considerable improvements in their service, starting with the physical condition of their roadbed and track, and working up, and even then, of course, can never compete on even terms.

It is often pointed out, in discussing the gauge, that these narrow-gauge lines have done very well, that their capitalization is small and, therefore, there is no reason to change, but, as will be shown later, it seems to the writer that they must look forward to the growth of the country and be prepared to meet it by modern economic means of transport or be crowded out. By the time they have provided adequate terminal facilities in the large cities, their capital expenditures will not be so much less than that of the broad-gauge lines, and they can never, of course, handle the same amount of business.

To convey some idea of the importance of the broad-gauge lines, Figs. 9, 10, and 11 show parts of the lines of the Southern, Pacific, and Central Argentine Railways, where they enter the City of Buenos Aires over expensive steel and masonry viaducts. They have extensive and expensive terminals, both passenger and freight; the Southern has four tracks for some distance out of the city. Fig. 12 shows the Southern Railway Passenger Station. The Central Argentine has nearly completed its second track to Rosario; the Central Argentine and Western have plans under way for the electrification of their suburban zones; the Central Argentine is building an extensive new terminal and station; the Western is building a tunnel connection to the Port Railway through the heart of the City of Buenos Aires, and nearly all have a considerable portion of their main lines stone-ballasted with stone at \$2 to \$3 (gold) per cu. m., f. o. b. cars (\$1.50 to \$2.50 per cu. yd.).

The Provinces of Entre Rios and Corrientes, within the confines of which are located the two principal medium-gauge railways, are almost entirely surrounded by the Rivers Parana and Uruguay. They are thus practically cut off entirely from communication with the rest of the Argentine, or indeed with any other part of the world, except by water transportation. Their isolated position, therefore, has rather retarded their development, in spite of the richness of the lands in many parts, and the railways have been merely local lines from the interior to the small ports along the rivers.

About 1906, however, it was decided to effect a connection with the City of Buenos Aires by an extension of the line to Ibicuy and a car ferry across the delta of the Parana River to Zarate, a small town in the Province of Buenos Aires. Here connection was made with the Central of Buenos Aires Railway, then a rural steam tramway of medium-gauge, over which running rights were obtained into the City of Buenos Aires, though at a point a fairly long distance out from the center. This through line was opened to public service only about 4 years ago.

At about the same time, also, the Central Railroad of Paraguay, which at that time was a broad-gauge line running from Asuncion, the capital of that country, to Villa Rica in the interior, was changed

Fig. 9.—Old Steel Viaduct, Central Argentine Railway, Entering Buenos Aires.

Fig. 10.—Masonry Viaduct, Pacific Railway, Entering Buenos Aires.

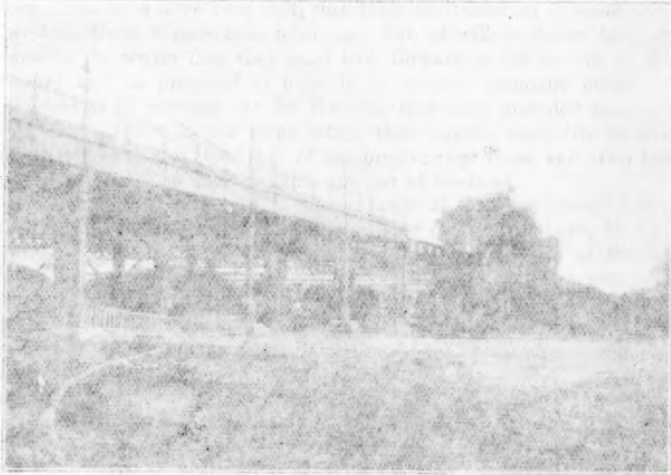


FIG. 9.—OLD STEEL VIADUCT, CENTRAL ARGENTINE RAILWAY, ENTERING BUENOS AIRES.

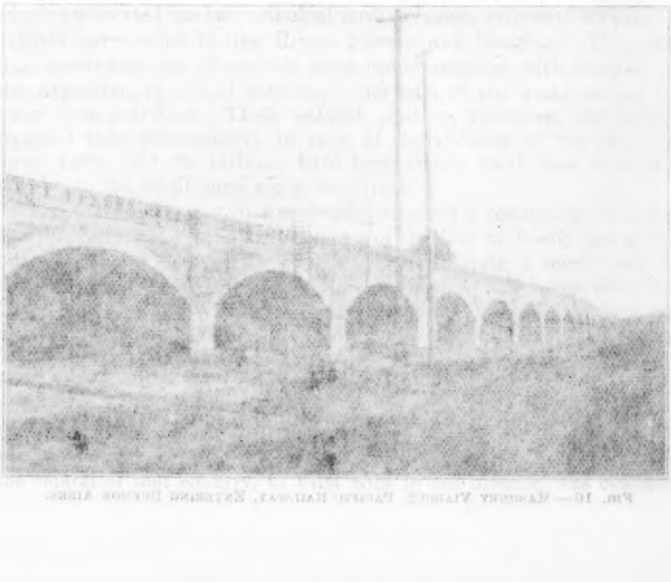


FIG. 10.—MASONRY VIADUCT, PACIFIC RAILWAY, ENTERING BUENOS AIRES.

FIG. 11.—STEEL BRIDGE AND MASONRY VIADUCT, CENTRAL ARGENTINE RAILWAY. ENTERING BUENOS AIRES.

FIG. 12.—SOUTHERN RAILWAY PASSENGER TERMINAL STATION, BUENOS AIRES.

to medium-gauge and extended to Encarnacion, opposite Posadas in Corrientes, and another car ferry established there, so that by these two car ferries, through train service between the City of Buenos Aires and Asuncion, through the Provinces of Entre Rios and Corrientes is now possible, thus putting all this section in close touch with Buenos Aires and the outside world.

It will be seen, therefore, that the whole railroad situation in the Argentine has undergone a radical change during the past 5 years or so. Previous to that time, the four large broad-gauge systems controlled by British interests, which in many respects were identical, dominated almost exclusively the transportation of the country, and the lines of other gauges were of little importance. Now the medium-gauge lines are not only linked together, but are all (except the Central of Buenos Aires) controlled by mutually friendly interests with a through route some 900 miles long from Buenos Aires to Asuncion. The narrow-gauge lines are linked up into not more than three systems, all of which are working together in harmony and may come under a single control. They reach the northern section of the country and the one in which the great development of the next 20 years is to take place. The question is, shall this development be by means of a system of narrow-gauge lines, can such a system offer adequate transportation facilities, and can it offer competition to the broad-gauge lines, should they decide to extend their lines into this section, and with river transportation on the Parana?

There is also to be considered the question of international connections, and the present and rapidly increasing interest in South America, both by Europe and North America, makes this important. To the writer the idea of a Pan-American Railroad as a continuous trunk line for the transportation of through freight and passengers from the United States and Canada to the extreme south, has always seemed almost chimerical. The variation in gauges, as will be seen, is an important obstacle, and such a line can never hope to compete for long-distance transportation through a tropical climate and across the vast and lengthy stretches of the most rugged mountains in the world, with the facilities offered by ocean steamers.

It is easy, of course, to foresee that at some time in the future, which seems now not a little distant, the various lines of railway will be linked together, and that there will be continuous connections. This, however, will not be, at least in the writer's estimation, until there is a local demand and local business sufficient to warrant the construction of the various constituent parts. It is also entirely probable that such a line will be useful and will be used for international communications, between any two or more countries within reasonable limits, and for this reason uniformity of gauge should be advocated throughout the continent and through the Central American zone,

this question being one which should be most energetically pressed at any further Pan-American Congresses and by the representatives of the Pan-American Union.

It has been previously pointed out how, in the Argentine, the four big broad-gauge systems radiating from Buenos Aires dominated the situation up to within 5 or 6 years ago, and how, since then, the other systems have formed coherent groups with connections to the National Capital. During the same period, also, an international syndicate of bankers has acquired control of the railway lines in the three States of Southern Brazil, and a substantial interest in those in the southerly half of the State of São Paulo. This same syndicate also has operating control over three lines in the Argentine, the Rosario á Puerto Belgrano, 5 ft. 6-in. gauge, the Central Cordoba, meter-gauge, and the Entre Rios, medium-gauge, it has established a community of interest with the North East Argentine Railway, which connects the Entre Rios Railway with the Paraguay Central Railway, which latter it also controls. It has also a substantial interest in the Antofagasta Railway, in Chile and Bolivia. In addition to its railways, this syndicate is taking great interest in the exploration, exploitation, and development of the countries served by its lines and those beyond into which they may be extended.

One of the lines controlled by these interests has already been pushed across the State of São Paulo, crossing the Alto Parana nearly 1 500 miles above its mouth, and, eventually, without doubt, will be pushed ahead through the State of Matto Grosso into Eastern Bolivia and to Sucre on the headwaters of the Amazon. Another line is projected along the boundary line between the States of Parana and Sta. Catharina, following the Valley of the Iguazu to its confluence with the Alto Parana, crossing the latter at a point some 500 miles below that of the line into Matto Grosso, just referred to, and connecting with the line now under construction eastward from Villa Rica on the Paraguay Central. This will give a through route from Rio, São Paulo, and all points in Southern Brazil to Asuncion, the capital of Paraguay, though there will be a break of gauge at the border. From Asuncion this line will probably be extended northwestward between the Pilcomayo and Paraguay Rivers toward Sucre.

There is another east and west line still farther south in Brazil from Porto Alegre to Uruguayana, a point on the River Uruguay opposite Paso de los Libres on the North East Argentine Railway, where connection can be made either to Santa Fé or Resistencia on the west side of the Parana, and from these latter places to all points in the northern section of the Argentine and to Bolivia. This latter route at present involves more or less delay, steamer transportation across the two rivers, and two breaks of gauge, but eventually this will be developed into a through route.

There is already through rail connection between Rio and Montevideo, with breaks of gauge, however, at São Paulo and at the border of Uruguay and Brazil. At least two schemes have been proposed for a short through line from Colonia, a point in Uruguay opposite Buenos Aires, to Rio, over which passengers can be carried in express trains at reasonably fast speeds, and thus shorten the length of the sea trip between the Argentine and Europe, and the time between Buenos Aires and Rio by nearly half. This, if it is ever realized, means a standard-gauge line through Southern Brazil, as the Uruguayan gauge is fixed and it would be out of the question to get the necessary degree of speed and comfort on any lesser gauge to compete with the fast, luxurious ocean steamers in which the trip can now be made and which are being improved every day. Perhaps present conditions hardly warrant the construction of this line as yet, but that it is a development of the not far distant future is hardly open to question.

There is, as is perhaps well known, one line which now crosses the Andes, giving direct rail connection between Buenos Aires and Valparaiso. This, however, involves a change at the break of gauge at Mendoza in the Argentine and at Los Andes in Chile, the Buenos Aires and Pacific is 5 ft. 6-in. gauge, the Transandine in the Argentine and Chile is meter-gauge, and the Chilean lines with which it connects are 5 ft. 6-in. Another connection is proposed in the south by the extension of the Southern Railway's (5 ft. 6-in.) line from Neuquen, and there is a proposal (which, however, it does not seem will be carried through in the immediate future) to build a line from Salta in the Argentine to Mejillones in Chile. Farther north, the Antofagasta Railway has been building an extension southeastward from Uyuni toward Tupiza, and probably before long it will be connected with the lines of the Argentine Government (meter-gauge) at La Quiaca.

Turning now to the Argentine: The area west and south of Bahia Blanca, it seems, must inevitably be developed by the Southern and Western Railways, that is, by the 5 ft. 6-in. gauge, and the country north of Santa Fé and Cordoba is the part in which may be expected the most important developments of the future which will be of interest to this discussion.

The line of the Central Cordoba Railway from Cordoba to Tucuman, and the Government line from Tucuman by Salta to Embarcacion may be taken as marking the dividing line between the mountains and the vast Argentine plain. Between Santa Fé and Tucuman there are two lines, the more southerly being the Central Argentine broad-gauge, and the more northerly the Central Norte, meter-gauge, operated by the Government. It is the area north of this latter line and east of the line from Tucuman to Embarcacion, and known generally as the Gran Chaco, in which the development of new territory by new

lines will probably be most active in the Argentine. The topography of this region is almost as blank as the space shown on the map, and its character does not change for at least some distance north of the Pilcomayo in Paraguay and Bolivia. The distance from Santa Fé to the northerly border of this region is about 750 miles, and from Asuncion to Tucuman about 550 miles, and it is probably safe to assume that railways may be built at a minimum cost and in practically straight lines over almost any portion of it.

Santa Fé is a port at present available at lowest river stages for ocean-going steamers drawing 18 ft. of water, and this depth will probably be increased in the near future. It seems to the writer improbable that there will be a port for ocean-going steamers established north of this point, so that the commerce of this vast region must come to Santa Fé. It is true that there may be some competition by way of the rivers, but experience elsewhere has shown that this will not be a serious matter for properly equipped railroads, though it might be for lines any less efficient than those designed in the light of the best present knowledge of the art. This area must be served by railroads of some sort, they must be efficient if they are to pay and be able to compete with water transport.

The Argentine Government has commenced the development of this region by lines running more or less westward from points on the Parana River at Resistencia and Formosa, apparently with the idea of bringing the products from the interior to the river and thence by fluvial transportation to the coast, the merchandise and supplies returning the same way. This, the writer believes to be fundamentally wrong, and that this area might better be developed by a series of north and south lines from Santa Fé and points on the existing lines between Santa Fé and Tucuman, which will reach up into the north beyond the Pilcomayo and bring all this vast region in touch with Buenos Aires. Here it may perhaps be well to refer again to the importance of the City of Buenos Aires as a distributing point for all merchandise, its dominating influence over the whole country as the seat of the national government, and the absolute importance of proper and adequate means of transportation to reach it from all parts of the country.

A certain number of east and west lines between Brazil and the north, along the lines already being developed, and as already indicated, will be necessary of course for the interchange of products and for communication between the large populations which will, in all probability, occupy this area before another quarter of a century has passed, but the line of least resistance to and from the southern part of eastern Bolivia, western Paraguay, and the Chaco is by the most direct north and south routes between this section and Buenos Aires.

The exact details of the lines along which the future development of this area may proceed, however, are not of particular importance in this discussion. The outline of the general scheme which has been presented will at least tend to show its close resemblance to that of the United States during the time our great West was being opened up to agricultural developments. Knowing as we do the important role played by the railroads in this era, it seems obvious that a system of transporation less efficient than our own in moving bulk freight long distances at low cost would handicap the development of the great area in the Southern Hemisphere.

DISCUSSION

Mr. Henry.

PHILIP W. HENRY,* M. AM. SOC. C. E.—This paper contains a great deal of valuable information, particularly for those interested in the construction and operation of railways in Latin America, and the author is to be congratulated on the exhaustive and intelligent manner in which he has treated this important subject. Of special value is his emphasis on the distinction between a "narrow-gauge railway" and a "light railway", which are often used as synonymous terms, although, as the author points out, a "light railway" may be of any gauge.

The speaker agrees with Mr. Lavis that the difference in the construction cost between a narrow-gauge and a standard-gauge railway is generally exaggerated. In a country the traffic of which does not admit of the construction of an expensive railway, and where the prevailing gauge is 4 ft. 8½ in. or greater, the speaker believes that it is better to cut the construction cost by raising the maximum grade and curvature rather than by lessening the gauge of the railway. Though the author treats of railways chiefly in the southern part of South America, more particularly Argentina and Southern Brazil, both lying within the temperate zone, where conditions are quite similar to those in the Central Western States, and where traffic will follow railway construction almost as rapidly as it did in those States, it must be remembered that in tropical South America and in Central America traffic does not follow with the same rapidity, and, therefore, the type of railways for those countries may be quite different from that suitable to Argentina and Southern Brazil. Broadly speaking, it is safe to assume that in Central America, the West Indies, and in tropical South America, the gross earnings of a new railway for the first few years will not be much greater than operating expenses, let alone interest on the capital invested, and that the development of traffic will be very slow, unless the parties building the railway undertake at the same time to develop traffic. Even in a highly fertile tropical country, it is seldom that the natives, who are well satisfied with their existing life, will make any effort to develop traffic on any considerable scale. The development of tropical agricultural traffic in most cases demands both capital and skill. This applies particularly to the raising of bananas and sugar cane, two of the most profitable tropical products.

If the railway is built in the high altitudes of Peru or Bolivia, not susceptible of agricultural development, it is essential that mineral traffic be developed, which, again, requires large investment of capital. In some cases, the building of a railway may be followed by the investment, on the part of outside capitalists, in the agricultural or mineral resources of the region traversed by the railway; but, gener-

* New York City.

ally speaking, if the parties building the railway are also its operators, it is essential for best results that, at the same time they acquire the concession for building the railway, they should take steps for the development of traffic—a point which is generally overlooked. The Governments often recognize this slow development of traffic, either by guaranteeing the bonds of the railway or by paying a cash subsidy approximating, if not equalling, the cost of construction. Mr. Henry.

The speaker believes that the author has over-estimated, if anything, the additional cost of constructing a standard-gauge railway (4 ft. 8½-in.) over that of a meter-gauge, especially where he adds an extra cost for "Contingencies, Expenses of Promotion, and Interest during Construction", amounting in all to 15%, which, according to his figures, equals $249 per mile on roads of light earthwork and bridging, $430 per mile on roads of medium earthwork and bridging, and $1 359 per mile on roads of heavy earthwork and bridging. It appears to the speaker that this item of general expense is the same, regardless of the gauge of the railway. In other respects, the speaker believes that the author's estimates are, if anything, more than ample. The extra cost of standard-gauge, given by him in detail for the three classes of railways, neglecting the 15% for general items as above, is as shown in Table 28.

TABLE 28.

	Light earthwork and bridging.	Medium earthwork and bridging.	Heavy earthwork and bridging.
Earthwork	$310	$980	$5 010
Bridges	150	500	2 000
Culverts	150	300	1 000
Ties	770	770	770
Laying and surfacing	100	100	100
Yards and sidings	170	170	170
Cattle guards, fences, etc.	10	10	10
Total for construction	$1 660	$2 830	$9 060
Less saving in equipment	100	100	100
Net Total	$1 560	$2 730	$8 960

To confirm these figures, it is interesting to compare the actual cost of building a railway of light earthwork and bridging in Bolivia, from Viacha to Oruro, 127 miles in length, with which the speaker was connected in 1906-09. This railway was of meter-gauge, roadbed of 5 m., the average cross-section for the entire length showing excavation actually paid for, both in cuts and borrow-pits, amounting to an average depth of 3.7 ft. Assuming that the railway had been of standard-gauge, instead of meter-gauge, it would have been necessary to handle 6½% additional earthwork. It is assumed that the cost of bridges,

Mr. Henry. trestles, and culverts, and of tracklaying and surfacing, would also have been increased by 6½%, certainly a liberal estimate. The ties, California redwood, costing about $2, delivered on the work, were 7 ft. in length, instead of 8 ft., if standard-gauge had been adopted, making an additional cost of 14²⁄₇% for ties. Table 29 shows the actual cost per mile of the different items affected—in the speaker's opinion—by the change in gauge, and the additional cost for each item in case standard-gauge had been adopted.

TABLE 29.

	Actual cost, meter-gauge.	Estimated additional cost, standard-gauge.	Total estimated cost, standard-gauge.
Grade	$4 200	$272	$4 472
Bridges, trestles and culverts	2 600	168	2 768
Tracklaying and surfacing	1 600	97	1 697
Ties	5 100	737	5 837
Total for these four items	$13 500	$1 274	$14 774

This shows an estimated additional cost of standard- over meter-gauge for a railway of light earthwork and bridging of $1 274, as compared with the author's estimate of $1 560, neglecting general expenses.

The question of gauge, except perhaps in Argentina and Southern Brazil, is not likely to arise on railways of medium or heavy earthwork and bridging, but it is more likely to arise on those railways built in districts of light traffic, where it is necessary to cut down the expense of construction to the lowest possible limit. As stated before, the speaker prefers to do this by building a railway of the gauge standard to the district, with high maximum grades and curvatures, say, 3½ or 4%, and 25° curves, intelligent discrimination being used in the application of these maxima. By using the Shay or a similar type of locomotive, the maximum grade may be run up to 8 per cent. As one train each way per day—sometimes only two or three trains per week—will take care of all the traffic, it is evident that the cost of conducting transportation is a small item when compared with the cost of maintenance of way and with the interest on the cost of construction. Therefore, under such conditions of light initial traffic and slow development of traffic, the first cost of construction, with due regard to maintenance of way, should be the controlling factor. At first sight, this would indicate a narrow-gauge railway, which admittedly can be built at from $1 000 to $1 500 per mile lower cost than standard-gauge, the interest on which at 7% is from $70 to $105 per mile per year, quite an item for a railway the gross earnings of which may run from $1 000 to $1 500 per mile per year. Still, other considerations must be taken into account.

such as gauge of connecting railways or of railways likely to be constructed. In Bolivia the gauge of the railway from Viacha to Oruro, 127 miles, was fixed at 1 m., as it was intended eventually to connect it with the Argentine Railway on the frontier, 425 miles distant, which is of meter-gauge, thus furnishing a through route from La Paz, the capital of Bolivia, to Buenos Aires. In Haiti and Santo Domingo, where the largest mileage of railways is of 42-in. gauge, that gauge will naturally be adopted for future railways in that island. In a country, however, where standard-gauge now prevails, or is likely to prevail, the building of a narrow-gauge railway is of doubtful expediency, and the speaker would prefer reducing the first cost by high maximum grade and curvature. Mr. Henry.

JAMES J. HILL,* F. AM. SOC. C. E. (by letter).—In the development of much of the territory of the United States, the original cost of construction of the 4 ft. 8½-in. gauge was all that (and in many cases more than) the country could bear, though the writer has long felt that it is not certain but that, for very heavy traffic, a 5 ft. 6-in. gauge would have been better. The difference in the dead weight of the car would not be great, and the center of gravity would be nearer the rail. Mr. Hill.

T. A. CORRY,† M. AM. SOC. C. E. (by letter).—Mr. Lavis' figures show that more than 60% of the Argentine Railways are broad-gauge, and only about 8% are standard. This fact, taken in connection with the lack of ballast and consequent difficulty in keeping the track in good surface, proves unquestionably that if only one gauge is to be adopted, the broad-gauge is the proper one for the Argentine, for Mr. Lavis is quite right in his remarks on the relative stability of broad- and narrow-gauges. Mr. Corry.

For such mountainous countries as Bolivia, Peru, Ecuador, Colombia, and Venezuela, the advantages of the standard gauge over the meter-gauge may be questioned for three reasons:

First. With the meter-gauge the capacity of 95% of the railways required, for the last 30 years and the next 30 years, to handle the available traffic would be quite ample.

Second. Sharper curves can be used on narrow-gauge than on wide-gauge for the simple reason that the rigid wheel base is shorter on both cars and locomotives for the narrower gauge, and the possibility of using sharper curves very greatly reduces the graduation item—a heavy one in traversing the slopes of the Andes, on either the west or east side, in the countries mentioned.

Third. It is good business to use a small gun and small shot for small game, especially when expenses are reduced thereby.

* St. Paul, Minn.
† Arequipa, Peru.

Mr. Corry. It is axiomatic, of course, that a considerable amount of traffic, even in a mountainous country, can be handled at less cost per ton per mile on a standard-gauge than on either of the narrower gauges, and the standard is the more economical, even if it does cost more to construct.

As to speed on curves of 100 m. radius (328 ft.), the safe limit is about 28 miles per hour, with the practical elevation slope of 6°, and this speed can be made without difficulty on either 3-ft. or meter-gauge. To refer to a concrete example, take the section of the Peruvian Southern Railway from Mollendo, on the Pacific, to Puno, on Lake Titicaca (Plate X). This road is of standard gauge, 326 miles in length, and was built in 1868-73, at a cost of 42 000 000 soles (at that time the sol was worth 48 pence, or, say, 97 cents). The maximum gradient was 4%, and the minimum radius of curve 100 m. (328 ft.), and the minimum was used very frequently.

The Mollendo Division is 172 km., or 107 miles, in length, and cost 12 000 000 soles. Besides numerous short tangents, it has six of from 2 to 7 miles in length, aggregating 25 miles. On the remaining 82 miles there are 421 curves, an average of 5 curves per mile. More than half these curves are of only 100 m. radius with an angle per mile of more than 250 degrees.

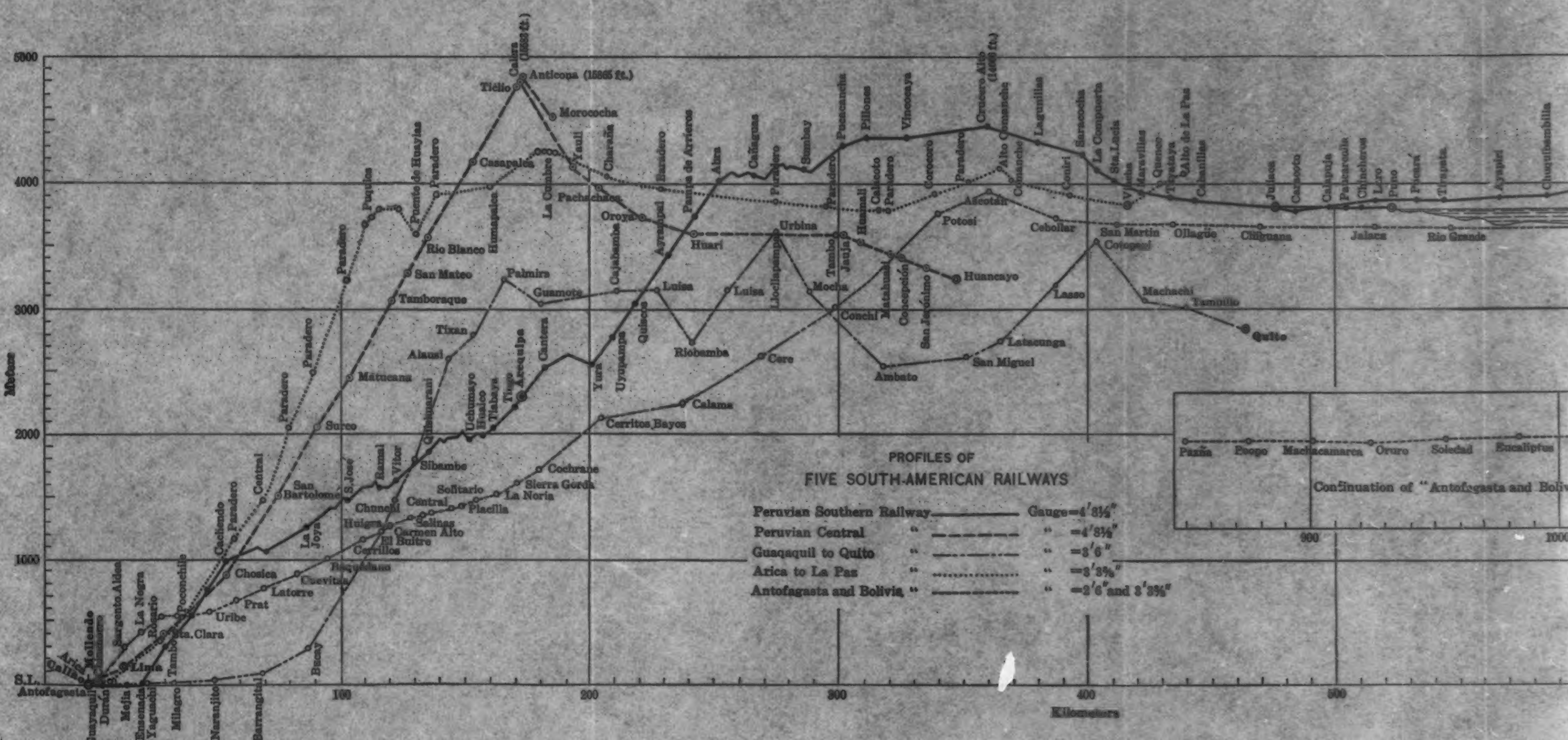
The Puno Division is 352 km. (219 miles) in length, and cost 30 000 000 soles. It has twelve tangents varying in length from 2 to 12 miles, comprising 46 miles, exclusive of those of less than 2 miles in length. The curve section is 173 miles in length and has 783 curves, an average of 4½ curves per mile, the larger number of which are on rough ground, and principally solid rock.

The total daily average tonnage, both ways, has never exceeded 600 in any of the 41 years of the road's existence. The average haul is large, being some 200 miles.

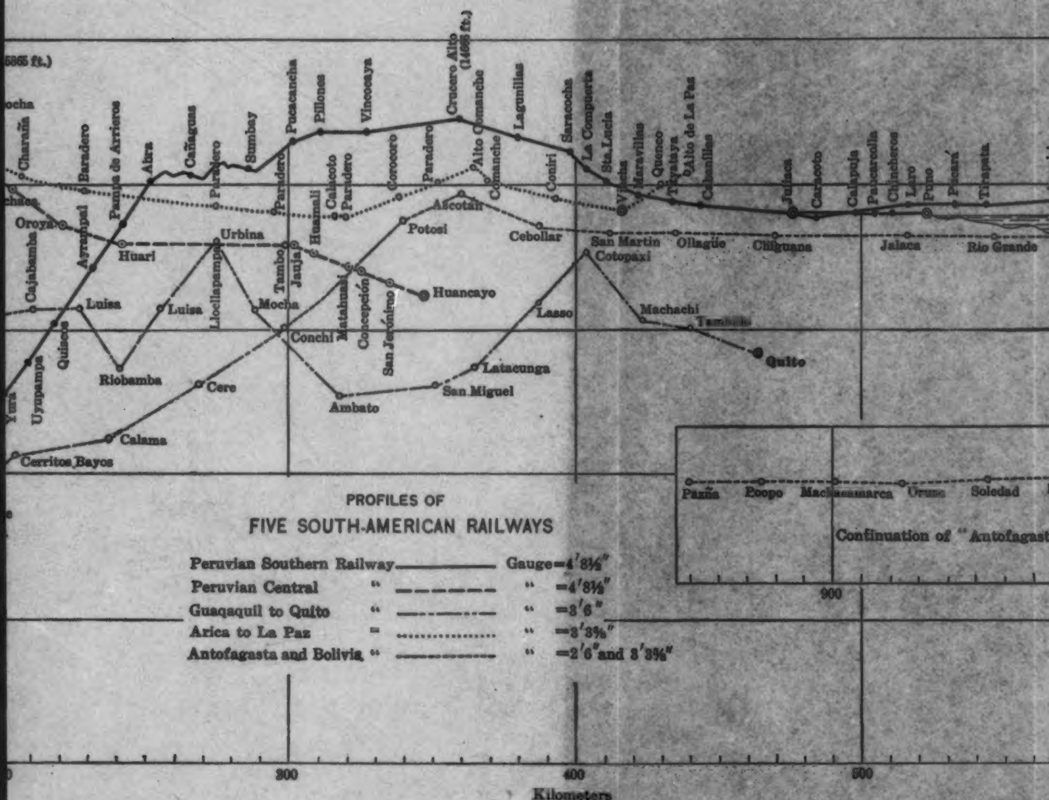
Whether it might have been advisable to have built this road with a 3-ft. gauge, using curves of only 80 m., is debatable on the ground of smaller first cost.

To give another illustration: A few years ago the writer was instructed to make a survey from a point on this road to a river of the Amazon Valley, less than 300 miles in length. The bases given were: maximum gradient, 3%; minimum radius of curve, 120 m.; and gauge, 3 ft. On these bases, the estimated cost reached the sum of $12 500 000, which was a prohibitive one for the traffic and advantages expected.

A resurvey of the roughest part (60 miles), using a gauge of 30 in. instead of 3 ft., with a minimum radius of curves of 50 m. and a gradient of 5% (for use with Shay locomotives), resulted in a reduction of $3 500 000, or nearly 30 per cent. The principal



PROFILES OF
FIVE SOUTH-AMERICAN RAILWAYS
Peruvian Southern Railway Gauge=4′8½″
Peruvian Central " =4′8½″
Guaqaquil to Quito " =3′6″
Arica to La Paz " =3′3⅜″
Antofagasta and Bolivia " =2′6″ and 3′3⅜″
Meters
Kilometers
5000
4000
3000
2000
1000
S.L.
100
200
300
400
500
900
1000
Continuation of "Antofagasta and Bolivia
Anticona (15865 ft.)
Ticlio
Morococha
Casapalca
Rio Blanco
San Mateo
Tamboraque
Matucana
Surco
Chosica
Lima
Callao
Oroya
Huari
Urbina
Huancayo
Jauja
Palmira
Guamote
Tixan
Alausi
Sibambe
Riobamba
Ambato
Latacunga
San Miguel
Machachi
Tambillo
Quito
Arequipa
Mollendo
Juliaca
Puno
Cerrillos Bayos
Calama
Cere
Conchi
Ascotán
Cebollar
San Martin
Ollagüe
Chiguana
Jalaca
Rio Grande
Cochrane
Sierra Gorda
La Noria
Placilla
Salinas
Carmen Alto
Baquedano
Cuevitas
Latorre
Prat
Uribe
Sta. Clara
Antofagasta
Duran
Milagro
Naranjito
Barranglal
Bucay
Pazña
Poopo
Machacamarca
Oruro
Soledad
Eucaliptus



PROFILES OF
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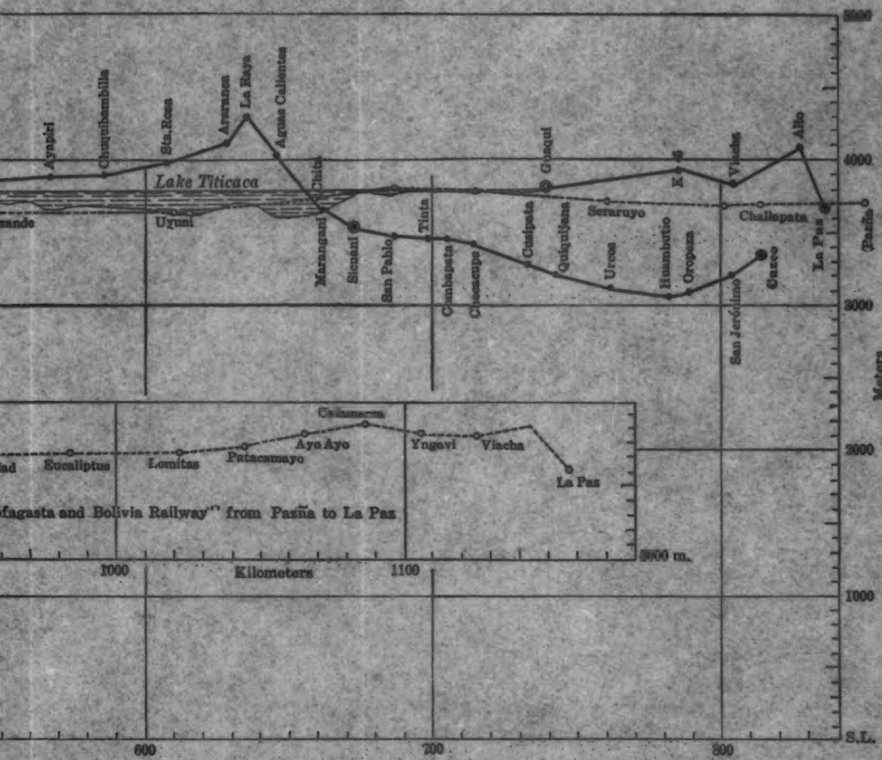
Guaqaquil to Quito " " =3′6″
Arica to La Paz " " =3′3⅜″
Antofagasta and Bolivia " " =2′6″ and 3′3⅜″
Crucero Alto (14666 ft.)
Charaña
Baradero
Pampa de Arrieros
Alto
Cañaguas
Paradero
Sumbay
Pucacancha
Pillones
Vincocaya
Comanche
Lagunillas
Saracocha
La Compuerta
Sta. Lucia
Maravillas
Quenco
Alto de La Paz
Caballitas
Juliaca
Caracoto
Calapuja
Paucarcolla
Chincheros
Puno
Tirapata
Oroya
Huari
Urbina
Tambo
Jauja
Huamali
Potosi
Ascotan
Cebollar
San Martin
Ollagüe
Chiguana
Jalaca
Rio Grande
Cajabamba
Luisa
Riobamba
Llocllapampa
Mocha
Conchi
Concepción
San Jerónimo
Huancayo
Cotopaxi
Lasso
Machachi
Quito
Latacunga
San Miguel
Ambato
Cere
Calama
Cerritos Bayos
Uyupampa
Pazña
Poopo
Oruro
Soledad
Continuation of "Antofagast
300
400
500
900
Kilometers

PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1817.
CORRY ON
THE GAUGE OF RAILWAYS.



reduction was due to the sharper curves, by which much heavy work and all the tunnels were avoided. The expected traffic would hardly reach 300 tons per day for the next 20 years. Mr. Corry.

The Antofagasta and Bolivia Railway Company is changing its 30-in. gauge to 1 m., so that now there are in Bolivia somewhat more than 800 miles of railways—all of 1 m. gauge.

In 1912, the total tonnage handled, including company material, was 1 600 710, or nearly 4 400 per day, the average distance hauled being about 130 miles. In view of the great expense being incurred by this road in changing its gauge to 1 m., it is to be presumed that the relative merits of meter- and standard-gauge have been carefully considered.

Although the writer is not an advocate of narrow-gauge railways, it must be admitted that there are still some mountainous districts where their use, especially as feeders, is advisable, even if it should prove necessary after a number of years to widen the gauge, and possibly at the same time make other improvements in the matter of alignment, gradients, etc. To put it another way, it may often be inadvisable to build now, for traffic to be handled, say, 40 years hence.

Frank Foster,* M. Am. Soc. C. E. (by letter).—In his able paper on railway gauges in South America, Mr. Lavis puts too lightly to one side what appears to the writer to be the principal advantage of the narrower gauges. Mr. Foster.

The Argentine broad-gauge (5 ft. 6-in.) will not allow of shunting with main-line engines on curves of 200 ft. radius, and requires turn-outs with curves of 600 ft. radius for goods trains; and curves well maintained of less than 1 600 ft. radius are rough for main-line running.

In comparison, the meter-gauge stock can circulate on curves of 120 ft. radius, can use turn-outs with curves of 250 ft. radius for goods service, and curves well maintained of 1 000 ft. radius can be taken comfortably at full speed on the main line.

The writer does not wish to imply that broad-gauge stock could not be constructed to circulate under these conditions, for they certainly could, but always at the sacrifice of velocity.

Mr. Lavis quite correctly points out that the broad and standard gauges have enormous advantages in speed over the narrow gauges, but, in fairness, this principal advantage of the narrow-gauge, namely, normal velocity on curves of smaller radius, should be equally extolled. As this is, in the writer's opinion, the unique advantage of the narrow gauges, their use in the Argentine is only justified in mountainous districts, and is to be deplored in the plains, for, without any financial or physical advantages, they have bred costly transhipment

* Buenos Aires, Argentine Republic.

Mr. Foster. which can only be avoided by a duplication of expenditure, frequently unjustifiable.

Unfortunately, the original railways in the Argentine were constructed of the 5 ft. 6-in. gauge in preference to the standard 4 ft. 8½-in.; and although any idea of correcting this error now or in the future would be financially unsound, as the advantages could in no way compensate the outlay, yet the writer is of the opinion that Mr. Lavis in no way exaggerates the advantages of the standard-gauge as compared with those of the broad-gauge.

An examination of the minimum construction and maximum rolling-stock gauges of the broad-gauge of the Argentine (Fig. 3) shows them to be very similar in dimensions to those of the standard-gauge in the United States, which indicates that in the Argentine no use is made of the wider gauge.

On several occasions the advisability of enlarging the rolling-stock gauge has been under discussion between the officials of the railways and the Argentine Government Railway Board, but it has been resolved, for the reasons given by Mr. Lavis, that there was not sufficient advantage, either in locomotives, coach, or wagon stock, even though the cost of increasing the construction gauge was not prohibitive. For instance, the broad-gauge lines of the Argentine at that time had no tunnels, and there are very few over-bridges.

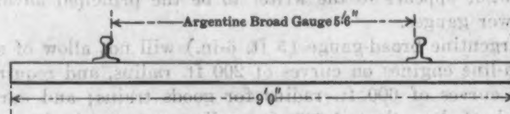


FIG. 13.

The carrying capacities of the Argentine broad-gauge and the United States standard-gauge are in reality the same, the only appreciable difference between them being the position of the rails, that is, the gauge.

The rail position of the Argentine 5 ft. 6-in. gauge, on 9-ft. sleepers, Fig. 13, represents a very distinct disadvantage to maintenance, as the length of the sleepers outside the rails is not sufficient for adequate support; theoretically, the maximum pressure transmitted by these sleepers is at their extremities instead of under the rails, as is the case with standard-gauge and 9-ft. sleepers, and is at that point 35% in excess of the average pressure, instead of 10 per cent. These disadvantages exist, to the writer's practical knowledge, and, in earth ballast, cause excessive pumping at the extremities after rains, and frequent breakage of the sleepers at the center.

In this respect the Argentine meter-gauge tracks, with 6-ft., and in some cases 6 ft. 8-in., sleepers, have better proportions, but the overhang of the rolling stock, shown by Fig. 3, is out of proportion—for

good maintenance on earth ballast—both to the gauge and to the sleeper length. This practically results in greater expenditure in maintenance, rougher riding, and reduced velocity, as compared with the broad gauge. Owing to the general absence of stone in the plains, the majority of the railways in the Argentine are earth-ballasted, except where the traffic is dense or in the few districts where stone exists. Mr. Foster.

The ratios between the rolling-stock gauge, length of sleepers, and gauge of track are given in Table 30.

The sleepers used in the Argentine are of a specially hard native timber (*Quebracho colorado*), of a density of 80 lb. per cu. ft., which permits the use of a dog-spike connection. These sleepers have a life equal to that of the rails they carry, in many cases exceeding 35 years.

Dense forests of these trees grow in the Northern Provinces of the country, and the soundest timber for sleepers is obtained from the branches sawn in two. The sawn side is laid downward for the bearing on the ballast and the upper side is adzed, as they are rough in section and all the sap-wood rots away entirely within a couple of years. A typical section of one of these sleepers, after the loss of the sapwood, is shown in Fig. 14.

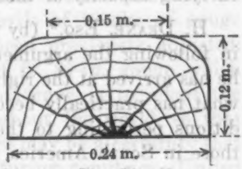


FIG. 14.

Sleepers can be obtained square sawn from the main tree trunk, but are not to be preferred, as they frequently split on exposure or spiking. Consequently, up to certain lengths, the cost per cubic foot is more or less constant, but for a length of more than 9 ft. the cost increases considerably.

As there are no native timbers other than the quebracho and similar hardwoods available in the Argentine, apart from the fact that legislation actually calls for the use of the timber of the country, everything is in favor of the use of quebracho in preference to any other timber on these railways.

TABLE 30.—RATIOS BETWEEN THE ROLLING STOCK, GAUGE, LENGTH OF SLEEPERS, AND GAUGE OF TRACK.

	Track gauge.	Sleeper length.	Rolling stock width.	Ratio of sleeper length to track gauge.	Ratio of rolling stock width to track gauge.	Ratio of rolling stock width to sleeper length.
Argentine: Broad-gauge.........	5 ft. 6 in.	9 ft. 0 in.	11 ft. 2 in.	1.63	2.03	1.24
Argentine: Narrow-gauge........	3 ft. 3⅜ in.	6 ft. 0 in.	10 ft. 6 in.	1.83	3.20	1.75
United States: Standard-gauge..	4 ft. 8½ in.	8 ft. 6 in.	10 ft. 3 in.	1.80	2.18	1.21
United Kingdom: Standard-gauge	4 ft. 8½ in.	9 ft. 0 in.	9 ft. 0 in.	1.91	1.91	1.00

Mr. Foster.

Therefore, the Argentine broad-gauge lines will probably continue to use sleepers of this length despite the serious disadvantage to maintenance indicated by the ratios given in Table 30.

On the level plains of the Argentine, curvature has not caused serious inconvenience thus far, but the waste of space in valuable terminals by the length of one-in-eight turn-outs (85 ft.) is an additional serious defect in the broader gauge, whereas, in the writer's opinion, there are no advantages of any kind in the use of the 5 ft. 6-in. gauge in the Argentine, as compared with those of the 4 ft. 8½-in. gauge.

The writer has not entered into the question of methods of transportation, but would point out that on the Buenos Aires Western Railway it is usual to run trains of more than 2 000 tons gross weight with locomotives weighing more than 100 tons. These trains at times attain a length of 2 500 ft. On that railway, wagons with carrying capacities of from 40 to 45 tons represent more than 80% of the total carrying capacity of those in service.

Mr. Deane.

H. Deane, Esq.* (by letter).—The writer has been much interested in following the arguments set forth by the author, and believes that he has arrived at the right conclusion when recommending for adoption what has practically become the standard and world gauge. The conditions pertaining to the problem in Australia differ somewhat from those in South America, in that the interests of the Australian States are really the same, and there is a tendency toward one or at least a similar kind of control in all matters. In the case of the railways there have taken place, for many years past, annual meetings of the officers and Commissioners who have charge in each State, and the desire always has been to work for one common object, the good of all. In South America there are many independent States, the Governments of which are not actuated by any desire for the common good; at the same time, it must be felt that individual interests would best be served if there were one gauge common to the whole country, for when the railways meet on the borders, interchange would thereby be facilitated.

In spite of what has been said in a few quarters as to the inadequacy of the 4 ft. 8½-in. gauge to meet all conditions of crowded traffic, it must be recognized that a medium gauge suitable for the vastly preponderating ordinary conditions should be adopted—one that is not so wide as to be prohibitive in cost when used in a country where the population is sparse and traffic is light. The 4 ft. 8½-in. gauge has been proved to meet all requirements fairly, and it is doubtful whether much more could be done with any much wider gauge than is achieved in the United States on the 4 ft. 8½-in.,

* Recently Engineer-in-Chief for Commonwealth Railways, Melbourne, Australia.

seeing that to get the full theoretical advantage of very much wider gauges, rolling stock would have to be built of very unwieldy dimensions, and this very unwieldiness would tend to reduce its economic value. Mr. Deane.

The case of India seems to show that a gauge much wider than 4 ft. 8½ in. would sooner or later prove a mistake. Railway construction was commenced in that country on a 5 ft. 6-in. gauge, but after a time that gauge seems to have been found to be inconveniently wide, and recourse was had to the meter-gauge for large extensions. The mileage of the railways on this gauge seems to be about 40% of the whole system. Had the 4 ft. 8½-in. gauge been adopted at the outset, there would probably have been no meter lines. Mr. Lavis has shown that, where the meter-gauge was thought necessary, the 4 ft. 8½-in. gauge meets the conditions quite as well. The writer proved that himself when constructing the Wolgan Valley Railway in New South Wales, and this has been mentioned by the author.

In selecting a gauge as a standard in any country, it should of course be borne in mind, that sooner or later connections will be made with outside countries, when all the evils of breaks of gauge will be experienced if the matter has not been previously adjusted.

There is one argument in favor of the 4 ft. 8½-in. gauge which should not be lost sight of. The present tendency is to adopt the same standards for mechanical details throughout the world, so that when an article is ordered from a distance there may be a certainty that it will fit into its place. The chief manufacturing countries—United States, Canada, Great Britain, and those of the continent of Europe—are engaged in building locomotives and other rolling stock to the 4 ft. 8½-in. gauge, and it is clear that if, in a certain country, there is a sudden demand for rolling stock which cannot be met out of its own resources, orders placed in any of the countries mentioned could be much more promptly met if the country ordering uses the same gauge; whereas, with a different gauge, there would necessarily be special designs, special patterns would have to be made, and so on.

G. F. F. Osborne,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has been connected with the administration of a railway comprising a section of 5 ft. 6-in. gauge adjacent to a large port, and a meter-gauge section, farther away, built as an extension of and as a feeder to the 5 ft. 6-in. section. Although the accounts and statistics of the two gauges were kept separately, both were worked as one system by one establishment of administrative and executive officers, and members of the staff in all grades were transferred, as occasion arose, without reference to the section on which they were serving or to serve. The results, therefore, afford a very fair basis for judging the relative operating merits of broad- and narrow-gauge for the conditions there obtaining. Mr. Osborne.

* Lucknow, India.

Mr. Osborne. Both lines are built over flat country intersected with waterways. There was very little cutting (practically none) on either system. The area served by the broad-gauge is densely populated; that served by the narrow-gauge more sparsely so. In both cases the individual inhabitant has a very small traffic value, either by his personal journeys or as a producer or consumer.

Before these lines were built, the country was served by a well-organized system of water carriage, which still exists; and competition is severe. The system has arrived at its present state partly by direct construction, but partly by the absorption, by purchase, of other lines; and their purchase price is now shown in the capital accounts as a lump sum. The broad-gauge section bears the brunt of the expenses due to terminals, has a heavy suburban traffic, and is generally constructed on a higher standard than the meter-gauge. On the other hand, the possible extension of the broad-gauge to replace the meter-gauge (now in progress over some 60 or 70 miles) has led to the construction of bridges having spans of 60 ft. and greater on the broad-gauge standard, and the expense (a considerable item) is charged to the meter-gauge. Another broad-gauge railway reaches the port by running powers over the quadruple portion of the broad-gauge section and two of the four tracks are necessitated by this traffic (which has not been here considered). The meter-gauge stops about 125 miles from the port, and all through traffic is transhipped at the point where the gauge changes.

The system is a network rather than a line, and has many branches. The longest distance run by a through train is 150 miles on the broad-gauge and 335 miles on the meter-gauge. Over the whole system the average haul of a passenger is 23 miles; of freight, 192 miles. Practically the whole staff is housed in buildings belonging to the administration and charged to the capital of the undertaking. Station buildings, houses, platforms, and similar structures are of masonry. The system is soundly built, well equipped, and adequately maintained.

Practically all the passenger stock is fitted with automatic brakes, but this is true of only a small percentage of the freight stock. The busier parts of both sections are worked on the block system, with token instruments; the remainder is worked on the written "line clear, authority to proceed" system. Signaling is normal danger.

The statistics in the tables are for 1912, and are derived from the half-yearly analysis of working the system; the sum or mean of the figures for the two half-years being taken. The error (if any) thus caused is not sufficient to affect the comparative value of the figures.

Before considering the operation statistics, it may be stated that the meter-gauge is being extended through country where the line is not expected to pay in itself for some time after opening, and profits have to be looked for in the increase in traffic brought thereby to exist-

ing portions of the system. The character of the traffic carried is shown by Table 32. Mr. Osborne.

TABLE 31.

Physical Characteristics.	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Mileage	512	1 117
Of which is Single-track	397	1 117
Double-track	94	
Quadruple-track	21	
Ruling gradient	1:300	1:200
Sharpest curve: Radius, in feet	000	500
Actual capital, cost per mile of line	$90 000	$35 500
Of which approximately is due to:		
Preliminary expenses and general charges	2 250	2 350
Rolling stock	21 750	6 725
Track, works and structures	66 000	26 425
Maximum weight of rail	90 lb.	50 lb.
Width of formation	20 ft.	16 ft.
Maximum permitted weight per axle of locomotives, in long tons	22½	10
Minimum seating space per passenger:		
Width	19½ in.	19½ in.
Floor space	3½ sq. ft.	3½ sq. ft.
Capacity	25 cu. ft.	25 cu. ft.
Standard ballast (stone) per foot of track	6 cu. ft.	3 cu. ft.
Cost per tie	$1.46	$0.56
Number of locomotives	253	238
Number of passenger vehicles	750	344
Number of freight vehicles	5 498	4 722
Number of stations on the line (including block cabins outside station limits)	152	209
Number of stations interlocked (including block cabins outside station limits)	69	24

TABLE 32.—PASSENGERS AND PRINCIPAL COMMODITIES CARRIED PER YEAR. (STATISTICS ARE NOT KEPT SEPARATELY FOR EACH SECTION.)

Tons are long tons of 2 240 lb.

Number of passengers	32 975 300
Fibers, raw (mostly loose or not fully pressed), in long tons	1 136 000
Coal, stone, etc	1 107 000
Grain (including flour) and seeds	570 000
Miscellaneous and general merchandise	580 000
Oils, drugs, chemicals, and spices	338 000
Metals and their ores	128 000
Fodder	114 000
Timber, wrought and unwrought	113 000
Salt	106 000
Sugar	84 000
Fibers, spun or woven	80 000
Railway material	72 000

The writer considers that the figures in Tables 32, 33, and 34 prove conclusively that the statement that a narrow-gauge line is more expensive to work than a broad-gauge is not of general application, but needs qualification, depending on the circumstances obtaining in each case. The writer admits the inferiority of a narrow-gauge to broader gauges in one particular only: "Lack of speed." If the speed on the narrow-gauge be sufficiently reduced below that on the broader

Mr. Osborne.

TABLE 33.—Operating Statistics.

Passenger Services.	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Passenger train-miles per year per mile of line	5 139	2 284
Earnings per passenger train per mile (includes mail and express)	$0.99	$0.85
Cost of working a passenger train 1 mile (includes mail and express)..............	0.58	0.42
Receipts per passenger-mile..............	0.0045	0.0043
Expenses per passenger-mile..............	0.0026	0.0021
Average number of passengers in a train..........	184	168
Average weight of a passenger train, in long tons..	264½	174½
Coal per passenger engine per mile, in pounds.....	58	39½
Average speed of passenger trains, including stops, in miles per hour..........	21¼	17⅛
Maximum speed of any passenger train, including stops, in miles per hour..............	35	25
Freight Services.		
Freight train-miles per year per mile of line........	3 482	1 583
Earnings per freight train per mile..............	$2.10	$1.35
Cost of working a freight train 1 mile..............	1.22	0.82½
Receipts per freight ton-mile, in long tons..........	0.0101	0.0107
Cost of carrying 1 ton of freight 1 mile, in long tons..............	0.0064	0.0065
Average load of freight in a train, in long tons.....	183	121⅛
Average gross weight of a freight train, in long tons..............	551½	352¼
Coal per freight engine per mile, in pounds..........	87½	62¼
Average through speed of a freight train, in miles per hour..............	14	12⅝
Ratio of paying freight to total capacity hauled....	34.44%	34.35%

Note.—The writer submits these figures for the purposes of comparison only, and not as the best results obtainable on either a broad-gauge or a meter-gauge.

TABLE 34.—General Statistics.

	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Cost of maintenance of track, works, and stations, per year per mile of track..............	$1 072	$462
Cost of maintenance of track, works, and stations, per year per train-mile..............	0.211	0.140
Fuel and all locomotive expenses (including wages and renewals), per train-mile..............	0.198	0.174
Car expenses, renewals, cleaning, oiling, etc., per train-mile..............	0.076	0.58
All other operating expenses, including station staff, per train-mile..............	0.194	0.116
Average cost per long ton of coal at engine shed.....	1.85	2.87
General charges: Audit, stores, medical, police, etc., per train-mile.............. } not separated { $0.055	....	....
Taxes, legal expenses, claims, payments to other lines, etc., ratio to gross earnings.......... } not separated { 5.85%	....	....
Average miles run per engine per day..............	73¼	67¾
Ratio of working expenses to gross earnings..........	60%	56%
Ratio of net earnings to total investment............	5⅓%	5⅛%

Note.—The writer submits these figures for purposes of comparison only, and not as the best results obtainable on either a broad-gauge or a meter-gauge.

gauge, all ill effects of curvature, stability, track stresses, maintenance, etc., are reduced to the level of the broader gauge, but it follows, partly as a result of this lack of speed, that increasing traffic will overtop the capacity of a narrow-gauge, single line, and demand further capital expenditure on doubling, conversion, or other means to increase its capacity, sooner than it will in the case of a single line of the broader gauge. Mr. Osborne.

In the particular case with reference to which the writer has given statistics, he considers that extensions on the meter-gauge were a mistake, not on account of any inherent defect of the meter-gauge itself, but because of the difficulties arising from the break of gauge. The cost of extending on the broad-gauge would probably not have involved an additional expenditure of more than $2 500 per mile, against which would have been the saving in rolling stock detained at transhipment junctions and the cost of transhipment terminals, probably sufficient in themselves to wipe out the whole $2 500 and leave the system to the good, as an operating proposition, by the absence of transhipment costs and the restriction to the development of traffic resulting from delay, loss, and damage to goods at the transhipment junctions.

If the system in question had been isolated, the meter-gauge over the whole system would probably have been as advantageous as the broad-gauge over the whole system; nor is it improbable that a 4 ft. 8½-in. gauge, with standards of construction similar to those of the existing gauges, would have obtained similar results. However, the question of the extensions and their gauge did not arise until the broad-gauge section adjacent to the port had been in operation many years, and the acceptance of the meter-gauge for the whole system was impossible for the following reasons:

1.—The lines which now comprise the system had not been brought under one agency at the time the meter-gauge was commenced, and the then diverse interests rendered impossible a complete system on that gauge.

2.—The broad-gauge was in existence, and its conversion would have been an unjustifiable expense.

3.—The port authorities have always very rightly opposed the entrance of mixed gauges to their docks, jetties, and warehouses.

This case is in some respects similar to the problem the author describes in the Argentine, and the writer holds that the correct solution would have been extension by lines of light construction on the 5 ft. 6-in. gauge, and is so far in agreement with the author's solution for the Argentine alone, namely, gradual conversion to 5 ft. 6-in.

Mr. Osborne.

There is considerable difference, however, between the selection of gauge for new lines and the question of converting existing lines to that gauge. In the first case, the extra expense amounts to little; in the second, it is a very important factor, and there must be considerable saving in the cost of operation to justify it. To the writer the solution appears to be: first, the construction of all future lines to the 5 ft. 6-in. gauge, then the conversion to broad-gauge of the more important narrow-gauge main lines, and afterward the gradual conversion of the less important narrow-gauge branch lines.

The writer, however, is not in accord with the author in his selection of the 4 ft. 8½-in. gauge for all lines in South America other than existing broad-gauge lines in the Argentine, which are to be left as they are.

It is not conceivable to the writer that the efficient, important, and financially strong broad-gauge companies of the Argentine, bound together with mutually friendly interests, will consent to remain as they are, and inactive, while they are being isolated and attacked by the 4 ft. 8½-in. gauge, and thus forced to convert to that gauge. It is much more likely that they will successfully seek to preserve their integrity by extensions opening up new territory, and then, on the Argentine will be thrown all the disablements of a mixed gauge which, in the writer's experience, is much more detrimental to the working of a railroad system as an efficient transportation machine than is the mere question of gauge, provided the same gauge be adhered to throughout. If the possibility (which the writer doubts) of a single gauge for South America be admitted, it should be either 5 ft. 6-in. or 3 ft. 3⅜-in., as the majority of the existing mileage is of those gauges.

The author's selection of a 4 ft. 8½-in. gauge for the whole of South America appears to be a compromise between the existing broad and narrow gauges, and based largely on the assumptions that the results obtained on the 4 ft. 8½-in. gauge of the United States and Canada show their railway systems to be the most efficient in existence, and that this efficiency is due, largely, if not entirely, to their gauge.

The writer, being familiar with the working of two large systems: one a 5 ft. 6-in. system which carries 200 long tons of freight 1 mile at a charge of $1.00 and pays 10% on the investment, the other a meter-gauge system, with only one-sixth the freight traffic density of the former line, which carries 111 long tons of freight 1 mile at a charge of $1.00 and pays 8⅓% on the investment, is inclined to question the first assumption, and, as to the second, without disparagement to those of faith and vision who found in a railway of 4 ft. 8½-in. (the then narrow) gauge an instrument for opening up the interior of the North American continent, or to those whose energy and resource-

fulness have enabled this gauge to cope with present traffic conditions and give the results to-day obtained, the writer's experience is that, under conditions now obtaining in this portion of North America, the 4 ft. 8½-in. gauge is a mistake, and that if the gauge were 5 ft. 6-in. the country would now be better served, and for reasons similar to those given by the author in casting the narrow gauge in favor of a broader one. Mr. Osborne.

The forces which have impelled the 4 ft. 8½-in. gauge in North America to economize by increasing the train load and so allowing revenue-ton-miles to increase while reducing revenue-train-miles are world wide, and where the 5 ft. 6-in., meter-, and other gauges have a live management, they affect them in the same way as they have affected the North American 4 ft. 8½-in. gauge, and South America will gain nothing that she would not otherwise receive along these lines by tearing up her existing 5 ft. 6-in. and 3 ft. 3⅜-in. gauges to replace them by 4 ft. 8½-in.

If the conditions and density of traffic in the area under discussion (Bahia Blanca to north of Rio) are ever likely to approach the conditions and density of traffic of the railways of the United States and Canada, then the extension of the Argentine 5 ft. 6-in. is undoubtedly the gauge that should be made universal.

The writer doubts whether such will ever be the case. There may be considerable similarity between conditions in the southern part of the United States and the southern part of this area, but at their northern extremities one is in the Tropical and the other is in the North Temperate Zone. So far as the writer's experience goes, the inhabitants of a tropical climate do not develop the vigor and capacity for intense effort that lead one to suppose they will ever develop a commerce sufficient to require and justify the railway facilities given in Northern Europe and the northern part of North America. The development of the meter-gauge in the tropical and the broad-gauge in the temperate portion of the area under consideration is not improbably an adjustment to the economic needs of each area, and the writer is inclined to look for the economic and economical development of these areas and their railways along the existing majority gauges, and would endeavor to consolidate the northern lines on the meter- and the southern lines, with the Argentine lines, on the broad-gauge, the dividing line between the gauges, in the absence of political and military considerations, being the traffic-shed between areas tributary to the ports of Buenos Aires and Rio, across which but little traffic would pass.

The writer's experience is that all gauges are good and capable of efficient operation where the standard to which the line is constructed is suitable to the character of the traffic to be moved, but that mixed

Mr. Osborne. gauges and breaks of gauge are an unmitigated evil. In the situation disclosed by the author, the needs would be met by preserving both the 5 ft. 6-in. and 3 ft. 3⅜-in. gauges, which comprise about 80% of the existing railway mileage, as each gauge could be confined to a definite area where the disabilities which arise from a break of gauge on a line of large through traffic would not occur.

Mr. Smith. JAMES ALEXANDER SMITH,* ESQ. (by letter).—The matter of the gauge of railways is of peculiar interest in Australia. It has been claimed, as an argument in favor of the 4 ft. 8½-in. gauge, that Australia has adopted this width, but this is not so. Some misunderstanding may have been caused by official publications issued by the Commonwealth Railway offices. These publications have been the subject of a Parliamentary enquiry, and it has been shown that they are inaccurate, and that the signatures of State Engineers, who have been consulted, are appended to matters in respect to which they did not concur, and that their protests were suppressed.

The writer suggests that, in the interests of accuracy, it be recorded that Australia has not, either in respect to the several States or the Federal Government, adopted the 4 ft. 8½-in. or any other uniform gauge. The facts, at the present date, are as follows:

Early in 1914 a conference of the Premiers of the several States —which control all the Australian railways in operation—and of the Commonwealth Prime Minister, was convened and held, to deal finally with the question of a uniform gauge for Australia.

Several of the States favored the appointment of an independent commission of railway experts, representing Great Britain, Europe and America.

The Prime Minister advocated the remission of the question to a lay "Interstate Commission," recently appointed to advise regarding trade and fiscal matters affecting more than one State. This was agreed to, and the conference disbanded.

Some months later, when interrogated in the Federal House, the Prime Minister stated that he had found that, under the Constitution, the Commission had no jurisdiction unless specially authorized by a vote of the House, therefore that the action just mentioned had been *ultra vires.* He also stated that he did not propose to seek the authority of the House, which has since been dissolved.

In the interim the Commonwealth Government has decided to reticulate the whole of the great "Northern Territory" with railways of 3 ft. 6-in. gauge. That is also to be adopted for the North to South Trans-Continental. Commitments on that basis are being entered into.

Simultaneously, the Commonwealth is constructing the East to West Trans-Continental on a 4 ft. 8½-in. gauge. This road connects, to

* Melbourne, Victoria, Australia.

the East, with the 5 ft. 3-in. system of South Australia and Victoria; and, on the West, in default of a specific arrangement by the Commonwealth as to gauge and financial support, Western Australia is preparing to construct its great extension of the Trans-Continental to the Coast on the 3 ft. 6-in. gauge. Mr. Smith.

The position is most deplorable, and indicates that unity of gauge is anything but an accomplished fact in Australia.

F. LAVIS,* M. AM. SOC. C. E. (by letter).—Some surprise has been expressed to the writer by certain engineers, who have had much experience in railroad construction, as to the necessity for any such elaborate discussion of the gauge question at this late day, and, even supposing the question to be of interest elsewhere, why the matter should be thought of sufficient interest to warrant the publication of such an elaborate paper by this Society. Mr. Lavis.

Of course, the latter criticism needs no argument for its defence, the Society is too large and important to confine its activities necessarily to matters of concern only to the United States, and the great interest which has been developed by the citizens of this country in South America since the presentation of the paper is ample justification, if any be needed, for the publication of any matter of interest in regard to the development of this vast area, in which the United States, and incidentally the engineers of this country, are bound to play an important part.

In regard to the first criticism, the writer admits that it must seem strange to American engineers, or to any others with knowledge of American railway methods, that it was considered necessary to go into so much detail in regard to many things which in the United States are accepted as axiomatic. A study, however, of the voluminous discussions of the past ten years and up to the present, in regard to the gauge question in India and Australia, two countries where the matter is now of vital interest, will show that most of the fallacies in regard to gauge—if not all—which we believed long ago disproved, are still held by many engineers as well as laymen by no means obscure. The persistence of the curve fallacy, as shown by some of the discussions, in spite of the concrete examples given in the paper, is only one instance of this.

The present status of the gauge question in Australia is shown by Mr. Smith's discussion, which apparently indicates that, in spite of a former decision to do all that was possible to unify the gauges, feverish activity seems to have developed in just the opposite direction, namely, to build as many lines as possible of different gauge.

Almost coincidently with the presentation of this paper, Sir Guilford L. Molesworth, one of the most prominent engineers connected

* New York City.

Mr. Lavis. with the administration of Indian railways, read a paper* urging the importance of the immediate consideration of some measures looking to the ultimate unification of the gauges of that country, but complicated by this question of gauge.

The present temporary stagnation of railway construction in South America is bound to be followed sooner or later by an era of activity. It seems to be probable that North American engineers will have a larger voice in these matters than they have had hitherto, and this seems especially appropriate inasmuch as the problems to be overcome are largely those which they have had to solve in the United States and Canada.

The writer is quite willing to admit that the criticism advanced by Mr. Henry is to some extent well taken, and that the estimates used in the paper of the difference in cost of construction of new lines due solely to difference of gauge, are "liberal" in so far as they exaggerate, if anything, the relatively higher cost of the wider gauge; but, inasmuch as these estimates were made with a view of their probable application to actual construction in a foreign country, it was hardly thought desirable to afford opportunity for criticism on the opposite side.

The problem of deciding between the narrow-gauge and 4 ft. 8½-in., or wider, does not seem to the writer to admit of much further discussion or doubt, but the matter of deciding between the 4 ft. 8½-in. and the 5 ft. 6-in., is one of considerable difficulty.

If the Argentine alone is considered, of course, it seems desirable for many reasons to adopt the 5 ft. 6-in., not because it has any greater capacity, but because it is established, and, for new extensions, its additional cost over the 4 ft. 8½-in. would be comparatively little. Mr. Hill's opinion in regard to this is worth noting; though a lowered center of gravity, as has been shown by the effect on track of some of the electric locomotives first designed, with very low centers of gravity, is not an unmixed blessing.

The writer's study of the possibility of applying some scheme of unification of gauge to the whole of that portion of South America south of the Valley of the Amazon (or even including part of its southern water-shed), on account of the political subdivisions and the apparent difficulty of coming to any agreement between so many diverse interests, may be considered rather fanciful. The fact, however, that as long ago as 1890, a commission headed by as eminent an engineer and railroad man as the late Alexander J. Cassatt, and working under an agreement participated in by practically every country in the Western Hemisphere, investigated the possibilities of constructing a line, or series of lines, of railway, linking up at least all the capital cities,

* "The Battle of the Gauges in India," *Journal*, East India Assoc., London, England, April, 1914.

shows that there has been for many years, in the minds of far-seeing men, a vision of a system of railroads covering the two continents, which will permit rapid and easy inter-communication between all parts. Considering, therefore, this vast area south of the Amazon and east of the Cordilleras as a single geographical unit to be covered by a network of railways, all of the same gauge and suited to modern conditions of transportation, such a project can hardly be regarded as entirely visionary. Mr. Lavis.

Whether it be possible finally to reach an agreement for this purpose is difficult to determine, but the writer believes that if it may be, that, for reasons given, the 4 ft. 8½-in. gauge, as Mr. Hill states was true of the United States, is all that the "traffic of the country could bear". The idea that the owners of the broad-gauge lines of the Argentine would not sit passively by without attempting to throw out extensions toward the north is one of course to be considered. The broad-gauge lines of the Argentine are to-day the most important group of transportation lines south of the United States, but this is only one item in a very complex problem which would have to be worked out to accomplish a desirable result, and, though not impossible of solution, it will require more than the efforts of any one man. The writer can only hope to have called attention to the problem and to some of the factors which will have to be considered in its solution, if a solution is ever desired.

The discussion by Mr. Corry is interesting in view of his long experience with the operation of the standard-gauge mountain roads of Southern Peru. Of course, it is true that railways should be built—as E. H. McHenry, M. Am. Soc. C. E., put it a long time ago—in such a way that the dollars spent for their construction will earn the most interest, and it is easily conceivable, and is admitted in the paper, that there are many mountain districts where narrow-gauge lines to be used as feeders may be built to advantage.

Mr. Corry shows that, on the standard-gauge lines of the Southern Peruvian Railway, curves of 328 ft. radius are successfully operated, and states that it is debatable whether it would have been advisable to have built the line with a gauge of 3 ft., using curves of 265 ft. radius. He states that it is axiomatic "that a considerable amount of traffic, even in a mountainous country, can be handled at less cost per ton per mile on a standard gauge than on either of the narrower gauges, and the standard is the more economical even if it does cost more to construct."

Yet Mr. Corry questions the advantages of the standard-gauge for mountainous regions, for the reason that sharper curves can be used and thus the graduation item can be reduced. This, in spite of the fact, as stated in the paper, that locomotives weighing 850 000 lb. are operated on the standard-gauge line of the main line of the Atchison,

Mr. Lavis. Topeka and Santa Fé Railroad on curves of 357 ft. (109 m.) radius, and the citation of the traffic handled on curves of 100 ft. radius on the rapid transit lines of New York City. For regions which may be served by lines of less than meter-gauge, the possibility of the use of excessively sharp curvature may perhaps be at times a controlling feature, but such lines can hardly ever be considered as main lines of transportation, and though they have their uses, and the Antofagasta is an excellent example of the value of 2 ft. 6-in. gauge, though it is now being forced to widen it, and will eventually be forced to widen again or double-track, they hardly enter into the consideration of the development of the transportation system of a continent.

Mr. Corry's illustration of the difference in cost between a line of 3-ft. gauge with 3% grades and curves of 120 m. radius, and one of 2 ft. 6-in. gauge with 5% grades and curves of 50 m. radius, can hardly be considered as having any bearing on the question of the difference in gauge, even as between 2 ft. 6-in. and 3-ft., inasmuch as the difference between the 3% and 5% grades might easily have caused all the difference in estimated cost.

It would be extremely foolish, of course, to say that standard-gauge, either 4 ft. 8½-in., or 5 ft. 6-in., should be used everywhere, and under every conceivable condition, and there is no question at all that Mr. Corry is right in pointing out that there are some places where it may be advisable to use a narrower gauge; but, as he also intimates, there is often an ultimate economy, by reason of the reduced operating expense of the wider gauge, even (and the writer believes, often more especially) on mountain lines.

Mr. Foster also seems to believe, at least to some extent, in the benefits to be derived from the possibilities of using sharper curves for narrow-gauge. This, perhaps, is due to some extent to the limitations imposed on the 5 ft. 6-in. lines of the Argentine, by reason of the construction of their rolling stock, as well as by Government regulations. Perhaps, however, when opportunity permits the further consideration, by the Government and the railways of the Argentine, of the desirability and necessity of the use of automatic couplers, this will eventually require the abolition of the side-buffers and the use of a coupling which will—as it does in the United States—permit the operation of much heavier rolling stock over quite as sharp curvature as that in use on the meter-gauge lines of the Argentine. It is true that, on the meter-gauge, well-maintained curves of 3 300 ft. radius can be taken comfortably at full speed, but this is also true of the 4 ft. 8½-in. gauge, and at much higher speeds, as shown by actual practice, and the writer believes that it would be equally true of the 5 ft. 6-in. gauge, with rolling stock of the type in general use in the United States. In regard to turn-outs for the use of main-line trains, and for general use in yards occupied by passenger trains, there has been a general

trend in the United States toward the use of No. 8 frogs (or 1 in 8) as a standard, in preference to those of greater angle. Mr. Lavis.

Mr. Osborne evidently has not read the paper carefully if he understands, as he states, that the writer claims that the efficiency of the railways of the United States is due entirely to the adoption of the 4 ft. 8½-in. gauge.

The mere fact that the writer suggests the adoption of the 5 ft. 6-in. gauge, when considering the Argentine alone, is sufficient to show that he considers efficiency with this gauge entirely possible and practical. All that is said in the paper, indeed, is that the 5 ft. 6-in., though costing more to build, offers no advantages as a transportation machine that the 4 ft. 8½-in. does not possess, nor does it permit of cheaper operation. There are doubtless many individual cases of railroads on which traffic is handled at a cost even lower than the average of the United States, but, speaking generally, it is true that, taking the country as a whole and considering the volume of traffic handled, the transportation of bulk freight is done at a lower cost here than elsewhere. If Mr. Osborne will read the paper carefully, however, he will note that it is claimed that this is due principally to the ability to handle very heavy trains, and has been brought about very largely by the increase in average train loading, in spite of great increases in cost of wages, etc. There is no claim in the paper that this economy, or perhaps more, might not have been attained on the 5 ft. 6-in. gauge, though, most assuredly, it could not have been attained on the meter-gauge. The claim is, however, that the extra cost of the 5 ft. 6-in. is not warranted, as it is not compensated for by any decrease in operating cost or any advantage over the 4 ft. 8½-in.

Mr. Osborne is quite mistaken in assuming that the area south of the water-shed of the Amazon is to any large degree within the tropics. Rio, which is taken as approximately the northern limit at the coast line, is just within the tropics, and though the interior part of the area under consideration runs somewhat farther north (that is, toward the equator), it is at an elevation of 2 000 ft. or more, and has a salubrious climate.

It is entirely improbable, as Mr. Osborne states, that the traffic of this vast area will ever approach the density of that of the United States, inasmuch as it is improbable that manufactured articles will be produced in it in any quantity, but nearly all the traffic is bulky and can only be hauled long distances cheaply—and it must be handled cheaply to move it—in cars of large capacity in comparatively heavy trains. The bulky agricultural traffic of India has been given by more than one prominent engineer and railway official as the reason for the adoption of the 5 ft. 6-in. gauge there, and though undoubtedly it is

Mr. Lavis. possible to handle heavier equipment and trains on the 5 ft. 6-in. than on the 4 ft. 8½-in., it is a fact that it is not done, so that it is considered that the 4 ft. 8½-in. is sufficient for these countries, and there is nothing to be gained by using the 5 ft. 6-in.

Time and time again the engineers and military authorities of India have advocated what Mr. Osborne does, that if it is not possible to unify the gauge, each gauge should be confined to a definite area; this has always been the cry, but it has never proved practical to do it, and the meter-gauge lines of Northern India are to-day agitating the matter of obtaining their own outlet to the coast, and Mr. Smith's testimony shows the effect in Australia. It is quite possible, of course, to admit—with Mr. Osborne—that under certain conditions all gauges are good and capable of efficient operation, but it is undoubtedly the consensus of opinion, of railway operators as well as of engineers, that the broad-gauge is a far more efficient transportation machine than the narrow-gauge, and enough more so in almost every instance to compensate for the comparatively very slight additional cost of construction, and as between the 4 ft. 8½-in. and 5 ft. 6-in. gauges, there is no particle of evidence to show that the former is not as efficient as the latter.

It is a fact that the 4 ft. 8½-in. gauge has served its purpose admirably, the effect of the introduction of a wider gauge anywhere has almost invariably been the subsequent introduction of the narrow-gauge, therefore, the writer feels that he is not only following well-established precedent in advocating the 4 ft. 8½-in. as the standard, if it be possible to unify the gauges of Southern South America, but also that this precedent is founded on wide practical experience.

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Paper No. 1318

GROUTED CUT-OFF FOR THE ESTACADA DAM.*

By Harold A. Rands, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. S. Howard Rippey, S. C. Hulse, Frank R. Fisher, H. L. Coburn, William H. Cushman, V. H. Hewes, Lazarus White, Frederic V. Abbot, J. F. Ramsbotham, Harrison Souder, Charles H. Paul, and Harold A. Rands.

Early in 1912, the Portland Railway, Light and Power Company, which is the principal public service corporation of Portland, Ore., completed what is known as the Estacada Hydro-Electric Development. This work, which includes an Ambursen dam, 90 ft. high, with power-house and transmission line, has already been described in several technical journals, and to redescribe these features is not the object of this paper. Inasmuch, however, as the treatment of the foundation material by the grouting process is the greatest thus far attempted, it is believed that a setting forth of this particular feature of the work will prove of interest to many engineers.

The location of the Estacada or River Mill plant, as it is sometimes called, is some 25 miles southeast from Portland, in the extreme western foot-hills of the Cascade Range, at the point where the Clackamas River, issuing from its long and tortuous canyon, begins its course through a rolling alluvial plain. The water-shed above the dam is approximately 800 sq. miles, and though the minimum flow for a short time in the dry months of August and September may fall to 700

* Presented at the Meeting of February 18th, 1914.

sec-ft., the average, of course, is much higher, and at times of extreme flood, like that of November, 1909, may rise to 40 000 sec-ft.

Previous Experience.—The Company was not without experience in construction work in these regions. For years it had operated a hydro-electric plant at Oregon City, where the Willamette River drops over a basaltic ledge with a fall of 40 ft. In 1906 it completed the Cazadero Station, 3 miles above the new Estacada plant. The operating head at this station is 130 ft., but the dam, which is 1¾ miles up the river from the plant, raises the water only 45 ft., the remainder being obtained by the flume, ditch, and dike by which the water is conveyed to the forebay. This dam is of logs, and though there is some seepage passing through and around it, this never has been excessive, and Mr. T. W. Sullivan, Hydraulic Engineer for the Company, who completed the work after it had been acquired in an incompleted state from other interests, states that no springs have at any time appeared, either in the bed of the river or in the vertical sides of the gorge, as a result of the dam.

In 1907 the Company began work preliminary to the construction of a dam about 150 ft. high at the head of slack water 2 miles above the Cazadero Dam. Here, under the supervision of S. C. Hulse, Assoc. M. Am. Soc. C. E., extensive borings were put down and tests of various kinds were made to determine the feasibility of erecting so high a structure on the formation found. These borings disclosed rock of so porous a nature that even a moderate pressure applied to the casings of the drill holes produced a leakage of many gallons per minute. Pressure applied to one hole would also cause the water to rise in other holes some distance away. The engineers had the benefit of the knowledge gained by the above construction and experience when in 1909 the Estacada site became the property of the Company.

Preliminary Borings.—At the site proper the river flows through a gorge of lava conglomerate, the sides of which are nearly vertical and from 50 to 70 ft. high. They are capped with a sloping over-burden of gravel, small boulders, and clay from 2 to 20 ft. deep. Preliminary borings, ranging in depth from 60 to 250 ft. and aggregating 2 500 ft. of drilling, were put down on both sides of the river for a distance of several hundred feet up and down stream, as well as on the island which later became a part of the dam.

That the general condition of the Estacada site would be similar to that at the site of the proposed 150-ft. dam previously mentioned was to be anticipated by one having a general knowledge of this region, of its geology, and of the great lava flow which now covers it. Concerning this flow the following is quoted from the Encyclopedia Britannica:

"This is probably the grandest lava-flow known to geology, covering as it does an area of about 200 000 square miles. Commencing in middle California as separate streams, in northern California it becomes a flood, completely mantling the smaller and flowing around the greater inequalities. In northern Oregon and Washington it becomes an absolutely universal flood, beneath which the whole original face of the country, with its hills and dales, mountains and valleys, lies buried several thousand feet. It covers the greater portion of northern California and north-western Nevada, nearly the whole of Oregon, Washington, and Idaho, and runs far into British Columbia on the north. The average thickness is probably not far from 2 000 feet, and the greatest (shown where the Columbia, Des Chutes, Snake, Salmon, and other rivers cut through it) about 4 000 feet. To produce this many successive flows took place, and a very long period of time must have elapsed during which the volcanic actions were going on."

Notwithstanding the universality of this great flood, the formations along those streams flowing westward from the Cascade Range with a rapid descent to the sea, as a rule, present less secure banks than do those traversing the high undulating plateau to the east, as even a casual observer who is familiar with the Snake or Des Chutes River Canyons, with their rim rock and columnar-like structure, may have noted. Concerning this western slope formation, Professor J. S. Diller, of the U. S. Geological Survey, in a report made in 1909 covering the site of the proposed 150-ft., Clackamas River Dam, expressed himself as follows:

"The rocks of the canyon walls are of four forms, volcanic breccias, lava sheets, volcanic dikes and terrace gravels, the volcanic breccias being most abundant.

"The conditions that confront the engineer along the Clackamas River in the volcanic breccia plain region are very much the same as will be found all along the western foot of the Cascade Range from the Columbia River in Oregon to the Feather River in California—one of the most important water-power belts in the United States—and the successful solution of the problem it presents at one point will greatly facilitate the work elsewhere."

To the foregoing, and possibly of greater interest to the engineer, may be added a statement from one, who, from an engineering standpoint, has probably a first-hand knowledge of the entire region second to no one. Certainly, in terseness of language and clearness of expression it leaves nothing to be desired. John F. Stevens, M. Am. Soc. C. E., in a report on the Estacada Dam Site, expressed himself as follows:

"The site of the proposed works lies in the west slope of the Cascade Range, and this range is well known to be, geologically taken, as of volcanic origin. Go where we may, for hundreds of miles North or South, we will find the upper valleys, canyons and river beds filled with what we call volcanic débris. Sometimes this volcanic matter appears as the hardest kind of basalt, sometimes as loose and friable as garden soil and generally occurs in every imaginable character and degree of solidity between the two, and it is not probable that nature has made any exception in this particular case. The limited number of test pits and borings which have been made show clearly that as far as the area they cover no well-defined, extremely hard masses of rock exist—that the rock is soft and shades away into clay, the latter, appearing under superficial examination much like a half burned brick. The whole formation to the non-geologically scientific man seems to be a mass of soft conglomerate, and this description is borne out in striking manner by the appearance of the nearly vertical walls of the canyon at the site of the dam and immediately adjacent."

Fig. 1, showing the left channel and vertical face of the island in the natural state, gives a good idea of the appearance of the rock. The pitted and scarred surface of the weathered breccia, with its hard irregular fragments embedded in the weakly cemented matrix of indifferent sand, is well shown. In some holes the formation, as shown by the preliminary borings, seemed to be much the same down to the bottom, and in others ledges of hard rock, from 5 to 20 ft. thick, were penetrated; in some the sandy matrix, most common in surface croppings, would change with depth to an even-textured clay, sufficiently compact to give a fair percentage of core; in several holes, though not among those put down on the site finally selected for the dam, this clay at considerable depth deteriorated to such a consistency that for many feet the diamond bit was replaced with a star bit, with which, rotating as did the diamond bit, very rapid progress was made for many consecutive feet.

The sandy matrix and, to a lesser degree, the clay-like matrix were ground away by the attrition of the drills and the harder fragments of

Fig. 1.—Left Channel. Weathered Breccia in Natural State.

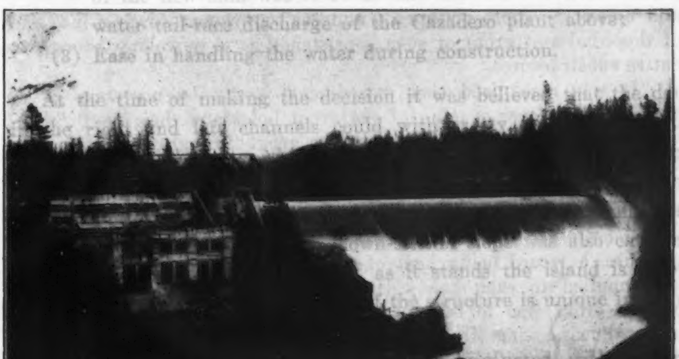

Fig. 2.—Dam, with Power-House nearing Completion.

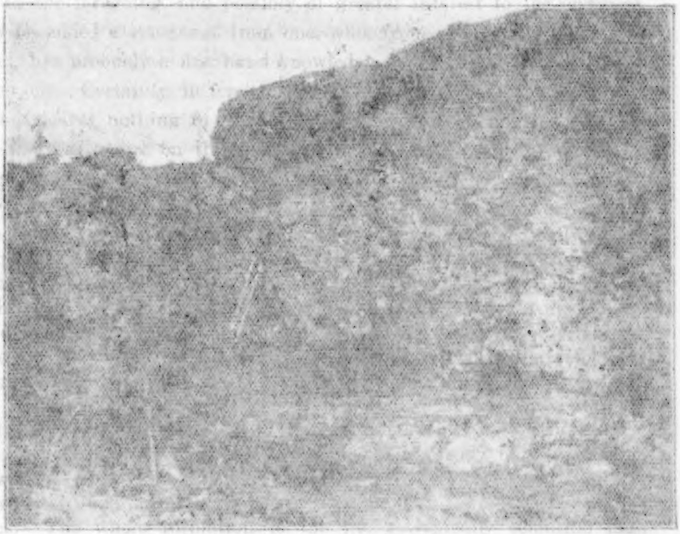

FIG. 1.—LEFT CHANNEL. WEATHERED BRECCIA IN NATURAL STATE.

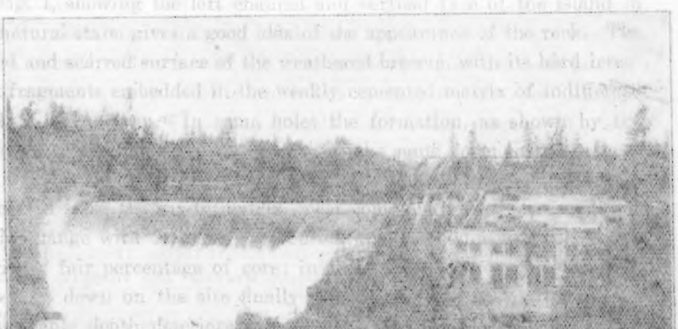

FIG. 2.—DAM, WITH POWER-HOUSE NEARING COMPLETION.

rock so that an average for the diamond bit, using the single-tube core barrel for the Sullivan diamond drill, operated by that Company's expert runners, was slightly less than 50 per cent. The double-tube barrel as used on part of the work gave core amounting to slightly less than 80 per cent. In these borings fragments of wood were passed through at different depths, and in subsequent excavation several stumps, one about 30 in. in diameter, and portions of logs in part petrified and in part in a semi-decayed state were encountered, so that altogether the term "débris" seems most fittingly to describe the material.

The Island Location.—A question constantly being asked by visiting engineers during construction was: Why was the dam located at the island rather than in the narrow gorge some few hundred feet up stream? The reply to this query is as follows:

In the beginning, three sites were under consideration, Site *A* at the railroad bridge, Site *B*, 200 ft. below Site *A*, and Site *C*, at the island. These are shown on Fig. 3. The reasons which led to the selection of Site *C* seem to have been:

(1) The soft clay encountered in the preliminary borings already mentioned;

(2) The great length of spillway which was essential, as the crest of the new dam was to be at the same elevation as the low-water tail-race discharge of the Cazadero plant above;

(3) Ease in handling the water during construction.

At the time of making the decision it was believed that the dam in the right and left channels could with safety be made to abut against the island. Later, this was deemed to be taking chances which amounted to gambling with fate, so that over the up-stream side of the island, cut to the slope of the dam, an Ambursen deck, minus most of the steel, was extended. The down-stream slope was also carpeted for a considerable distance, so that as it stands the island is incorporated in the dam, and this portion of the structure is unique in being "a hollow dam" filled with island.

The plan for grouting the foundation originated with Messrs. Sellers and Rippey, of Philadelphia, Consulting Engineers on the work. Their conception of the problem and what they hoped to accomplish is well set forth in a paper by H. V. Schreiber, M. Am. Soc. C. E., a member of the above firm. This paper was presented

at the Eighth Annual Convention of the National Association of Cement Users. Mr. Schreiber in part said:*

"After a careful study of the conditions disclosed by the investigation it was decided that in order to secure an impermeable foundation it was necessary to practically provide for actually changing the structural character of the underlying formation. To do this Mr. S. Howard Rippey, chief engineer of Messrs. Sellers and Rippey, engineers on the work, recommended that the most promising method would be the solidification of the porous material by the introduction of cement grout under pressure, but that the success of the work must be susceptible of demonstration by actual test before the superstructures should be started. Thorough inquiry failed to disclose any precedent for the use of grout for the general treatment of foundations, although it was found that cavities in limestone rock under the New Croton Dam had been filled with grout, much as a dentist would fill a cavity in a tooth. The grouting method had also been used in filling back of lining walls in tunnels, etc. Notwithstanding the absence of precedent, it was decided to proceed with a grouting scheme and a program was outlined for preventing leakage of water from the reservoir created by the dam, under or around the dam, to the low tail water level below the dam and the experiments which should be made to properly demonstrate the efficacy of the method before the complete development should be undertaken were prescribed. The general idea provided for drilling a double line of holes of an average depth of 50 ft. under the heel of the dam across the entire valley to and under the shore abutments and the subsequent forcing into each of these holes of grout of such consistency as to percolate through the entire substructure.

"It was recognized that experiments would be necessary to determine the proper spacing of the grout holes and their depth, which would insure sufficient diffusion of the grout through the varying material encountered to create a continuous impermeable barrier to prevent seepage of water from the reservoir under the hydrostatic head which would be created by the dam. After drilling the double line of holes, the program contemplated the test of each hole with water pressure, a record being kept of the quantity escaping and the pressure applied to each hole. Upon the completion of the tests, the cement grout was to be pumped into the holes under pressure and after allowing time for hardening, a third line of holes was to be drilled midway between the first two or outer lines and tested with water pressure. The idea was that if water pressure applied to the centre holes at or slightly above the hydrostatic pressure to which the rock would be subjected by the water in the reservoir after completion of the dam and no appreciable leakage occurred, it would be rea-

* *Engineering Record*, March 23d, 1912, p. 314.

sonably certain that the cement grout had proven effective in making the entire foundation impermeable.

The drilling for the grouting included 555 holes, aggregating 34 038 lin. ft. For a short time the Sullivan [illegible] drills, operated by steam, and also the Davis-Calyx shot drills, operated by electric power, were used. The diamond drills, being required at the site of the proposed 150-ft. dam, were removed after putting down 1 119 lin. ft., and from that time to the end the [illegible] drills were used exclusively. These drills are made by the Ingersoll-Rand Company, and those used on this work, with one exception, were the small "Class C-O" machine. These may be operated by [illegible] is a secondary consideration, but on this work each was driven by a 3-h.p. direct-current motor which [illegible]

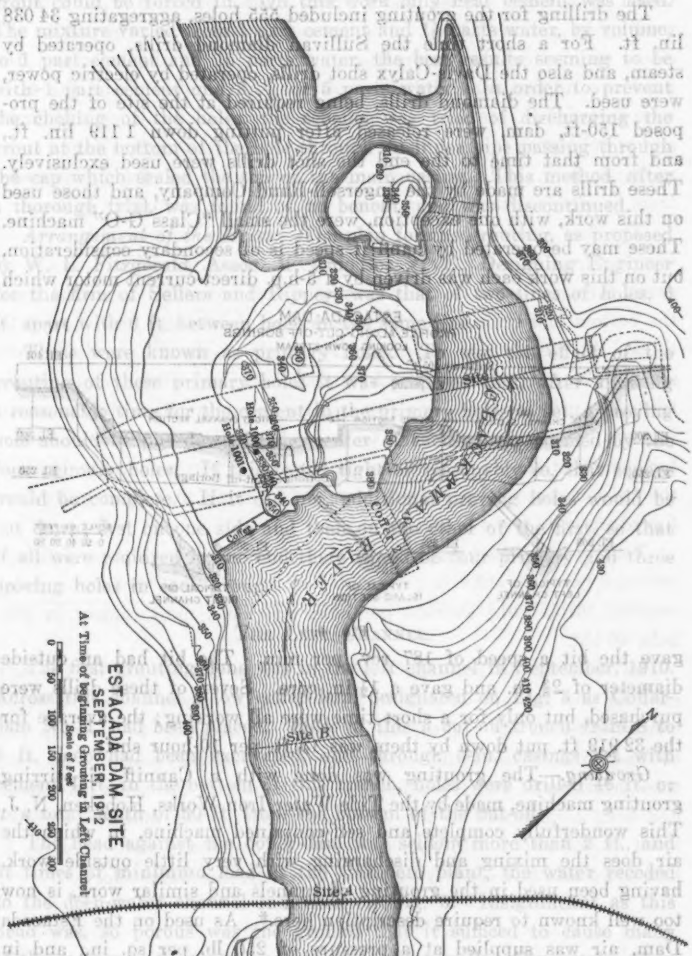

FIG. 3.

gave the bit a speed of 15[illegible] per min. The bit had an outside diameter of 2⅜ in. and gave a 1⅝-in. core. Seven of these drills were purchased, but only for a short time were all working; the average for the 32 919 ft. put down by them was [illegible] per 24-hour shift.

Grouting.—The grouting was done with a Canniff stirring grouting machine, made by the [illegible] Iron Works, Hoboken, N. J. This wonderfully complete and well-designed machine, in which the air does the mixing and discharging, needs very little outside work, having been used in the grouting of tunnels and similar work, is now too well known to require description here.* As used on the Estacada Dam, air was supplied at a pressure of [illegible] lb. per sq. in., and in

* One of these machines was sold late to the U. S. Reclamation Service, and used in grouting the Lahontan Dam of the Truckee-Carson Project. An excellent description of the machine, with drawings, will be found in an article by D. W. Cole, M. Am. Soc. C. E., in *Engineering News*, April 24, 1913, p. 647.

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The drilling for the grouting included 555 holes, aggregating 34 038 lin. ft. For a short time the Sullivan diamond drills, operated by steam, and also the Davis-Calyx shot drills, operated by electric power, were used. The diamond drills, being required at the site of the proposed 150-ft. dam, were released after putting down 1 119 lin. ft., and from that time to the end the shot drills were used exclusively. These drills are made by the Ingersoll-Rand Company, and those used on this work, with one exception, were the small "Class G-O" machine. These may be operated by hand if speed is of secondary consideration, but on this work each was driven by a 5-h.p. direct-current motor which

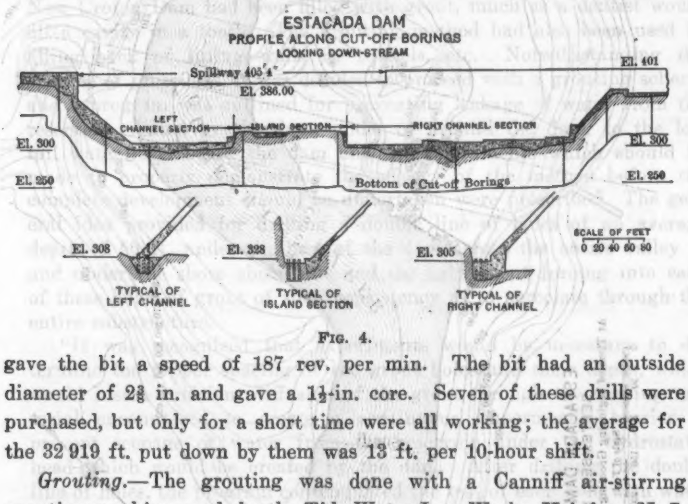


Fig. 4.

gave the bit a speed of 187 rev. per min. The bit had an outside diameter of $2\frac{3}{8}$ in. and gave a $1\frac{1}{2}$-in. core. Seven of these drills were purchased, but only for a short time were all working; the average for the 32 919 ft. put down by them was 13 ft. per 10-hour shift.

Grouting.—The grouting was done with a Canniff air-stirring grouting machine, made by the Tide Water Iron Works, Hoboken, N. J. This wonderfully complete and self-contained machine, in which the air does the mixing and discharging with very little outside work, having been used in the grouting of tunnels and similar work, is now too well known to require description here.* As used on the Estacada Dam, air was supplied at a pressure of 250 lb. per sq. in., and in

* One of these machines was sold later to the U. S. Reclamation Service, and used in grouting the Lahontan Dam of the Truckee-Carson Project. An excellent description of the machine, with drawings, will be found in an article by D. W. Cole, M. Am. Soc. C. E., in *Engineering News*, April 3d, 1913, p. 647.

grouting it was customary to start each hole at a pressure of 25 lb., or whatever was required to start the discharge, and as the hole tightened to increase the pressure, finally ending when at 200 lb. no more grout could be forced in. On this work only neat cement was used. The mixture varied from 1 part cement and $1\frac{1}{2}$ parts water, by volume, to 1 part cement and 15 parts water, the best results seeming to be with 1 part cement and 3, 4, or 5 parts water. In order to prevent the choking of the holes, the scheme was tried of discharging the grout at the bottom of the hole through an inner pipe passing through the cap which sealed the top of the main casing. This method, after a thorough trial, was found of no benefit, and was discontinued.

Arrangement of the Holes.—The plan for the grouting, as proposed by W. L. Fitzgerald, Assoc. M. Am. Soc. C. E., Designing Engineer for the firm of Sellers and Rippey, was that of two lines of holes, 6 ft. apart with 6 ft. between holes in the same line.

These were known as primary holes. To test the effect of the grouting of these primary holes it was proposed that, after allowing a reasonable time for the cement in the primary holes to set, a proving hole should be put down in the center of each square formed by the four primary holes. If this tested tight or reasonably so, that square would be complete. If it did not, additional proving holes would be put down, first on one side and then on the other of the first, so that if all were required in the end there would be four primary and three proving holes in each group.

The Left Channel.

The first grouting done was in the left channel in September, 1910. Across this channel a low coffer-dam, designated on Fig. 3 as Coffer-dam No. 1, had been placed. Back of this a cut-off trench from 8 to 9 ft. deep had been excavated, and through 3-in. casings set with cement 4 ft. in the bottom of this trench, holes were drilled 46 ft. or to a total depth of 50 ft. from the bottom of the cut-off.

The head against the coffer-dam was seldom more than 2 ft., and at times of minimum load at the Cazadero plant, the water receded to the deep-water channel indicated on Fig. 3. Insignificant as this head was, so porous was the ground that it sufficed to cause many of the casings to flow continuously as Artesian wells. Conversely, when pressure was applied to the casings of a number of the holes, leakage

was observed at several points in the river bed, which here was conglomerate bare of gravel, nearly out to the deep-water channel.

Unfortunately, for purposes of comparison, these first holes were tested by a pump taking water from a tank provided with a gauge reading in gallons direct. The pressure gauge was attached to the elbow connecting the water line to the casing of the hole, and the pressures as registered by this and the water taken from the tank were recorded at intervals of 1 min. Tests were made at one-half depth and at full depth for each hole. This method of testing was found to be unsatisfactory, and subsequent tests were made by gravity pressure from an elevated tank also provided with a gauge reading in gallons.

The first attempt to grout was made with the bottom of the cut-off trench bare, but so great was the loss of pressure and the escape of grout through seams in the rock that operations were suspended until the bottom had been protected with a mat of concrete from 2 to 3 ft. thick. At the time of beginning the grouting, though the drills were still working at that point, there had been put down some twenty testing or primary holes. So free was the communication that in grouting one hole it was necessary to cap the neighboring ungrouted holes for some distance in each direction, though it was customary to leave the caps of these holes slightly loose until the air and clear water first forced up gave place to the grout.

Twenty-five holes grouted at this time show results as follows: Average pressure, 19.4 lb.; average leakage, 85.7 gal. per min.; total cement, 167.25 bbl.; average per hole, 6.69 bbl.; most cement in any hole, 40 bbl.

As soon as the grouting was done across the bottom of the left channel, the urgency of getting to work on the power-house and other construction in the right channel made it necessary to blow up the coffer-dam, thus for a time ending operations at this point. This, too, was unfortunate, for, though it was not known at the time, operations had been commenced in the worst and most porous material encountered on the entire job, and, as future developments were to show, it had to be left but slightly better, apparently, for these efforts.

The Right Channel.

That the construction of the dam proper might proceed with the least possible hindrance from the drills, William H. Cushman, M.

PRIMARY HOLES GROUTED BEFORE CONCRETING

Hole No.	711	712	713	714	715	716	717	718	719	720	
Test Date	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/6'11	4/6'11	4/6'11	
Gall's per Min.	41	100	76	78	96	47	40	75	40	15	
Grout Date	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/20'11	4/10'11	4/21'11	4/24'11	
Bbls. Cement	7.75	5	1.25	25	6.25	6.25	9.25	2.5	1.25	10	
Hole No.	721	722	723	724	725	726	727	728	729	730	Average
Test Date	4/6'11	3/18'11	3/18'11	3/18'11	No	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	
Gall's per Min.	85	75	7	94	Test	4	1	15	30	9	46.2
Grout Date	4/21'11	4/11'11	4/10'11	4/21'11	4/12'11	3/19'11	4/10'11	4/10'11	3/20'11	3/20'11	
Bbls. Cement	3.25	1	3	2	3.75	1	2.75	3.5	1.75	6.25	5.14
Hole No.	511	512	513	514	515	516	517	518	519	520	
Test Date	4/21'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/6'11	4/6'11	4/6'11	
Gall's per Min.	80	81	90	69	95	40	43	55	79	20	
Grout Date	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/26'11	4/30'11	4/10'11	4/17'11	4/24'11	
Bbls. Cement	4.25	1	1	1	5	2.25	4	3.75	4.5	1.75	
Hole No.	521	522	523	524	525	526	527	528	529	530	Average
Test Date	4/6'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	
Gall's per Min.	40	91	93	34	26	93	80	83	13	5	60.75
Grout Date	4/24'11	3/21'11	4/21'11	4/21'11	3/19'11	4/12'11	3/20'11	3/20'11	3/19'11	3/19'11	
Bbls. Cement	5.5	6.5	2	2.5	1.25	3	1	2	3.75	1	2.85

INTERMEDIATE PRIMARY HOLES GROUTED AFTER CONCRETING

Hole No.	711a	712a	714a	715a	718a	719a	720a	721a	722a	
Test Date	6/7'11	6/9'11	6/13'11	6/9'11	6/15'11	6/21'11	6/16'11	6/15'11	6/20'11	
Gall's per Min	22	2	2.5	2	1	2.7	20	40	15	
Grout Date	6/7'11	6/9'11	6/13'11	6/9'11	6/15'11	6/23'11	6/17'11	6/15'11	6/20'11	
Bbls. Cement	11.25	1	1	16.75	1.5	1.25	1	1.75	1.25	
Hole No.	724a	725a	726a	727a	527a	728a	528a	729a	529a	Average
Test Date	6/16'11	6/21'11	6/28'11	6/16'11	7/2'11	6/21'11	7/6'11	6/17'11	6/28'11	
Gall's per Min	2.7	1.2	1.8	3.5	2.5	4.5	1	63	46	12.0
Grout Date	6/17'11	6/23'11	6/28'11	6/17'11	7/2'11	6/23'11	7/6'11	6/23'11	7/2'11	
Bbls. Cement	1.5	1	1.25	1.25	1	1	1.25	1	3	2.72

PROVING HOLES

Hole No.	911 b	913 b	915 b	916 b	918 b	919 b	921 b	923 b	925 b	927 b	928 b	929 b	Average
Test Date	6/25'11	6/9'11	6/7'11	8/16'11	7/17'11	6/20'11	6/25'11	6/26'11	7/5'11	7/8'11	7/13'11	7/8'11	
Gall's per Min	2.5	2	4.2	2.1	1.3	14.6	4	3.5	1.2	3	12.5	44.5	7.95
Grout Date	8/12'11	8/12'11	8/12'11	8/17'11	8/12'11	8/12'11	8/1'11	7/14'11	7/14'11	7/13'11	7/13'11	7/13'11	
Bbls. Cement	4.5	0.75	4	1.25	0.75	1.75	1.50	1	1.25	1.25	1.25	1.5	1.78

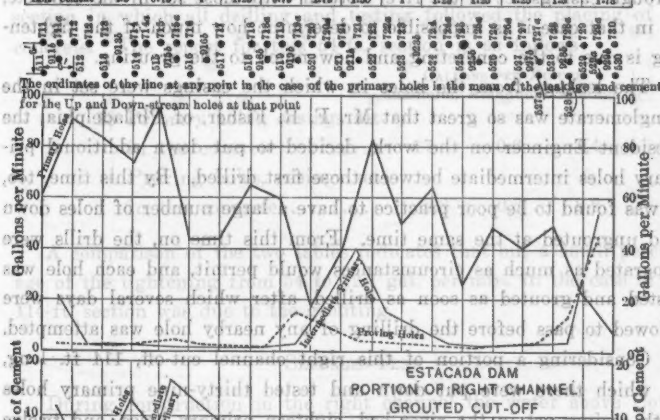


FIG. 5.

Am. Soc. C. E., representing the consulting engineers, directed that the holes be put down, not through the heel of the dam, as in the left channel, but so as to form a supplementary cut-off immediately up stream therefrom, the plan being to set the casings in the bare conglomerate of the river bottom just back of the cut-off proper, and later, after the grouting and proving were done, to hand-pick a trench to the depth of the casings, namely, 3 ft. or a little deeper. The breccia encountered was sufficiently soft, where speed was not of first consideration, to be removed in this way. This supplementary trench would then be filled with concrete which, bonding with the heel of the dam on one side and with its bottom resting on the curtain-wall supposed to be formed by the grout, would effectually prevent seepage under the dam.

The first grouting was done through casings set in accordance with this plan. However, as in the left channel, the surface material was so lacking in solidity that this method of procedure was not found to be satisfactory, and the plan was modified to the extent of excavating and filling the supplementary cut-off trench before drilling, which was then done through casings set in the concrete. It is apparent that, for purposes of comparison, it is improper to consider the holes tested through casings set in the river bottom with those set in the concrete, as in that case it is impossible to determine how much of the tightening is due to the concreting and how much to the grouting.

The leakage from the holes in which the casings were set in the conglomerate was so great that Mr. F. R. Fisher, of Philadelphia, the Resident Engineer on the work, decided to put down additional primary holes intermediate between those first drilled. By this time, too, it was found to be poor practice to have a large number of holes down and ungrouted at the same time. From this time on, the drills were separated as much as circumstances would permit, and each hole was tested and grouted as soon as drilled, after which several days were allowed to pass before the drilling of any nearby hole was attempted.

Considering a portion of this right channel cut-off, 114 ft. long, in which there were put down and tested thirty-nine primary holes with casings set in the river bed, eighteen primary holes, and twelve proving holes with casings set in the concrete, the following results are noted:

	Gallons per Minute. Maximum.	Minimum.	Average.
39 First primary holes, casings set in river bed....	100	1.0	53.7
18 Second primary holes, casings set in concrete.....	63	1.0	13.0
12 Proving holes, casings set in the concrete.........	44.5	1.2	7.95

The 39 holes first noted, it should be stated, were tested from a tank at Elevation 430, or 46 ft. above the crest of the spillway, and giving a mean head of 182 ft. on the bottom of the hole; the other tests in this, and in the Island and Left Channel Sections following, were made from a tank at Elevation 394, or 7 ft. above the spillway crest, and giving a mean head of 146 ft. on the bottom of the hole.

How much of the tightening, from an average of 54 gal. per min. for the 39 holes drilled and tested before the concrete was placed, to an average of 12.7 gal. per min. for the 18 holes drilled and tested after the concrete was placed, was due to the concrete can never be known. An idea of the relative effect of the concrete and the grout may be deduced by considering an adjoining though somewhat tighter section in which all drilling and testing followed the placing of the concrete. The report for this 54-ft. section is as follows:

	Gallons per Minute. Maximum.	Minimum.	Average.
18 Primary holes, casings set in the concrete...........	17	0.6	4.4
4 Proving holes, casings set in the concrete..........	2.8	0.8	2.2

A comparison of the two tables indicates that but a small percentage of the tightening from 54 to 12.7 gal. per min. in the case of the 114-ft. section was due to the grouting.

Seepage Test.

During construction in the right channel the water above Coffer-dam No. 2 stood at a mean elevation of 317 ft.; behind the coffer-dam, by using pumps, it was held at Elevation 295.

Near the middle of this channel a depression some 30 ft. wide was found in the conglomerate rock. On the cut-off line this was 15 ft.

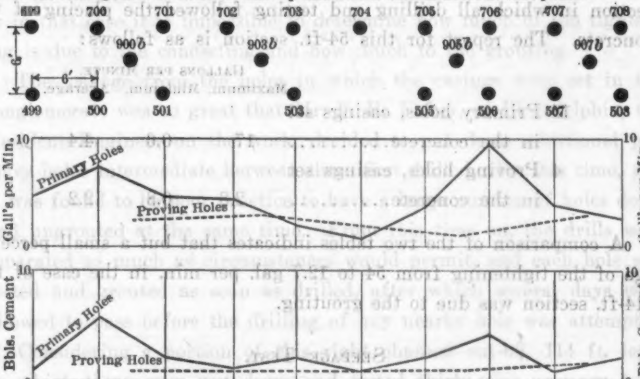
PRIMARY HOLES

Hole No.	699	499	700	500	701	501	702	703	503	
Test Date	6/4'11	6/26'11	6/2'11	5/26'11	5/3'11	6/13'11	5/28'11	5/31'11	5/25'11	
Gall's per Min.	4.4	4.7	2.5	11.9	5.8	3.2	4.3	1.2	2.5	
Grout Date	6/4'11	6/26'11	6/2'11	5/27'11	5/31'11	6/13'11	5/29'11	5/31'11	5/27'11	
Bbls. Cement	0.75	1.25	1	10.5	2.75	1.75	1.25	0.75	4	
Hole No.	704	705	505	706	506	707	507	708	508	Average
Test Date	5/27'11	5/30'11	5/28'11	5/24'11	5/26'11	6/5'11	6/22'11	6/6'11	6/25'11	
Gall's per Min.	0.6	3.2	3.6	17	1.7	3.3	4	1.7	3	4.4
Grout Date	5/27'11	5/30'11	5/29'11	5/25'11	5/26'11	6/5'11	6/22'11	6/7'11	6/25'11	
Bbls. Cement	1	0.75	3.75	7.5	1	1	1.25	1	3.5	2.486

PROVING HOLES

Hole No.	900 *b*	903 *b*	905 *b*	907 *b*	Average
Test Date	6/20'11	6/20'11	6/12'11	6/29'11	
Gall's per Min.	2.5	2.8	.8	2.7	2.2
Grout Date	8/24'11	8/13'11	8/13'11	8/13'11	
Bbls. Cement	1	1	1	1.75	1.187

The ordinates of the line at any point in the case of the primary holes is the mean of the leakage and Cement for the Up and Down-stream holes at that point



ESTACADA DAM
PORTION OF RIGHT CHANNEL
GROUTED CUT-OFF

FIG. 6.

deep, measured from the average level of the surrounding rock, but on the center line of the dam, 80 ft. down stream, this depth had increased to 35 ft. At some earlier age this depression had been worn smooth by the current, but at the time of building the dam it was

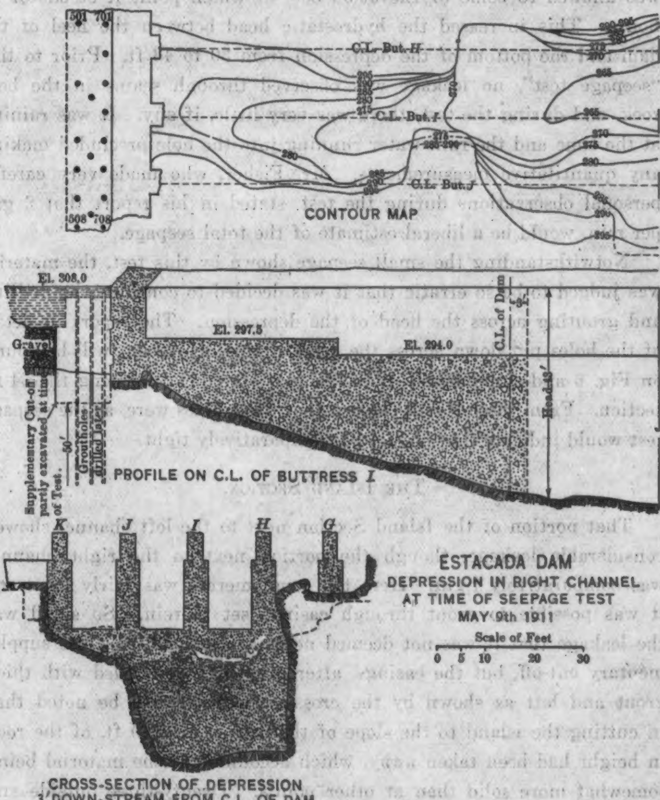


Fig. 7.

filled with loose sand, gravel, and hard boulders. This accumulation was removed, and, as a foundation for the buttresses at this point, mass concrete to a depth of several feet was placed over the entire bottom.

On May 9th, 1911, at which time the concrete had been placed in this gorge to a point 3 ft. below the center line of the dam, and when no drilling and grouting had been done along the cut-off line across the head of the same, the pumps were stopped and the water was allowed to come to Elevation 308, at which point it stood for 6¾ hours. This increased the hydrostatic head between the heel of the dam and the bottom of the depression from 30 to 43 ft. Prior to this "seepage test", no leakage was observed through seams in the bed-rock, and during the test, there was very little, if any. It was raining at the time and the rain water running into the hole precluded making any quantitative measurements. Mr. Fisher, who made very careful personal observations during the test, stated in his report that 2 gal. per min. would be a liberal estimate of the total seepage.

Notwithstanding the small seepage shown by this test, the material was judged to be so erratic that it was decided to continue the drilling and grouting across the head of the depression. The report in detail of the holes put down across the head of this depression will be found on Fig. 6 and summarized in the tabular statement covering the 54-ft. section. From these it will be seen that these holes were, as the seepage test would indicate they should be, comparatively tight.

The Island Section.

That portion of the Island Section next to the left channel showed considerable leakage, though the portion next to the right channel was comparatively tight. Here the conglomerate was fairly firm, and it was possible to grout through casings set therein. So small was the leakage that it was not deemed necessary to hand-pick the supplementary cut-off, but the casings, after grouting, were filled with thick grout and left as shown by the cross-section. It will be noted that in cutting the island to the slope of the deck nearly 50 ft. of the rock in height had been taken away, which accounts for the material being somewhat more solid than at other points. Twenty-three outside and six proving holes put down in this portion show leakage as follows:

	Gallons per Minute. Maximum.	Minimum.	Average.
14 First primary............	8.5	1.4	4.1
9 Second primary............	6.6	0.7	2.4
6 Proving	2.3	0.9	1.2

FIRST PRIMARY

Hole No.	666	466	667	467	668	468	669	
Test Date	5/10'11	5/13'11	5/15'11	5/8'11	5/21'11	5/17'11	5/9'11	
Gall's per Min.	2	1.4	4.2	4	4	1.6	2.5	
Grout Date	5/11'11	5/14'11	5/15'11	5/9'11	5/21'11	5/17'11	5/12'11	
Bbls. Cement	1	0.5	1.5	50.25	1	1.25	1.25	
Hole No.	469	670	470	671	471	672	472	Average
Test Date	5/13'11	5/16'11	5/15'11	5/9'11	5/9'11	5/13'11	5/13'11	
Gall's per Min.	3.5	6.5	3.5	8.5	5	4	7	4.1
Grout Date	5/14'11	5/16'11	5/15'11	5/14'11	5/13'11	5/14'11	5/13'11	
Bbls. Cement	1.25	10	2.5	3.5	1	2	19	6.85

SECOND PRIMARY

Hole No.	666 a	466 a	667 a	467 a	468 a	670 a	470 a	671 a	471 a	Average
Test Date	5/9'11	5/20'11	5/24'11	5/22'11	5/19'11	5/20'11	5/18'11	5/16'11	5/18'11	
Gall's per Min.	0.7	0.9	1.5	1.5	5.5	1.5	1.6	6.6	1.7	2.4
Grout Date	5/19'11	5/21'11	5/24'11	5/22'11	5/19'11	5/20'11	5/18'11	5/16'11	5/18'11	
Bbls. Cement	1	1.25	1.5	1.5	1.25	1	1.5	1.25	1.25	1.277

PROVING HOLES

Hole No.	866 b	867 b	868 b	869 b	870 b	871 b	Average
Test Date	6/5'11	6/5'11	6/8'11	6/8'11	6/12'11	6/12'11	
Gall's per Min.	0.9	0.9	1	1	1.3	2.3	1.2
Grout Date	8/31'11	9/5'11	8/31'11	8/31'11	8/30'11	8/28'11	
Bbls. Cement	1	1	1	1	1	1.25	1.04

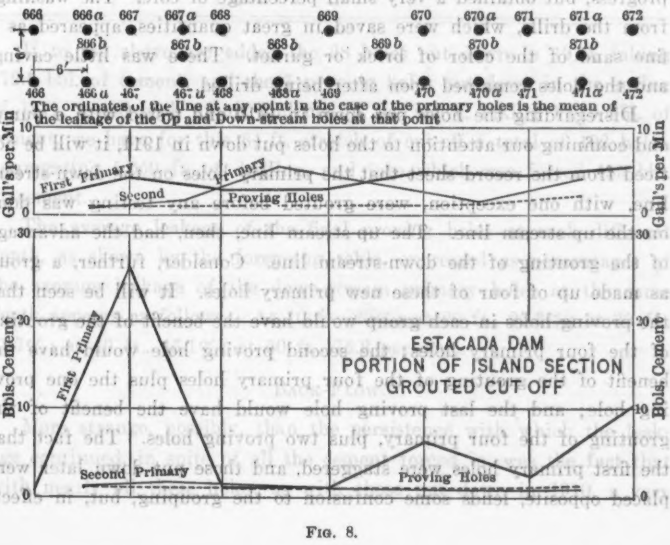


FIG. 8.

The same holes took the following quantities of cement:

	Barrels of Cement. Maximum.	Minimum.	Average.
14 First primary..........	50.25	1	6.85
9 Second primary..........	1.50	1	1.27
6 Proving	1.25	1	1.05

Return to the Left Channel.

In the spring of 1911 several proving holes put down in the left channel and tested with the pump, as had been the original holes, notwithstanding the great quantity of cement that had been forced in, showed leakage and intercommunication between holes practically the same as for the holes put down before any grouting was done. When this was discovered, Mr. Fisher decided to put down new primary holes between those drilled the previous fall. These were tested every 10 ft. as drilled. These tests were so remarkable that the result of the work at this point will be discussed in detail.

By referring to the profile it will be seen that at depths varying from 12 to 40 ft. below the concrete, the normal conglomerate gave place to a water-bearing stratum. Through this the drills made good progress, but obtained a very small percentage of core. The washings from the drills, which were saved in great quantities, appeared as a fine sand of the color of brick or garnet. There was little caving, and the holes remained open after being drilled.

Disregarding the holes put down in 1910 and tested with a pump, and confining our attention to the holes put down in 1911, it will be noticed from the record sheet that the primary holes on the down-stream line, with one exception, were grouted before any testing was done on the up-stream line. The up-stream line, then, had the advantage of the grouting of the down-stream line. Consider, further, a group as made up of four of these new primary holes. It will be seen that the proving holes in each group would have the benefit of the grouting of the four primary holes; the second proving hole would have the benefit of the grouting of the four primary holes plus the one proving hole; and the last proving hole would have the benefit of the grouting of the four primary, plus two proving holes. The fact that the first primary holes were staggered, and those put down later were placed opposite, lends some confusion to the grouping, but, in effect,

still disregarding the holes put down in 1910, the drills were passed over the ground five times. This being the case, the holes drilled in each successive passage should show decreased leakage. That this progressive tightening did not follow and that the results were somewhat complicated and disappointing is shown graphically by the curves of Figs. 9 and 10. Tabulated, the averages appear as follows:

	Average Leakage of Holes, in Gallons per Minute, at Depths of:				
	10 ft.	20 ft.	30 ft.	40 ft.	50 ft.
Down-stream primary	10.3	15.0	29.9	33.2	47.3
Up-stream "	8.05	4.3	14.7	47.2	56.9
First proving	1.4	3.9	15.5	50.7	68.6
Second "	1.6	5.1	13.7	22.1	31.7
Final "	2.0	3.8	6.5	15.0	35.6

Cement was forced into these holes as follows:

	Totals.	Averages.
14 Down-stream primary	117.75 bbl.	8.38 bbl.
13 Up-stream "	79.25 "	6.09 "
11 First proving	21.50 "	1.95 "
12 Second "	20.25 "	1.84 "
17 Final "	26.50 "	1.55 "
67 Holes in all	264.75 "	3.95 "

If to the above are added the 28 holes put down in 1910, taking 172.5 bbl. of cement, and the 7 proving holes put down in the spring of 1911, tested with pump, and not in the above, taking 17.75 bbl. of cement, we have for this 84-ft. stretch of cut-off a total of 102 holes, aggregating 5 050 ft. of drilling, and into which was forced 455 bbl. of cement.

The average leakage of the final proving holes at each different depth, as shown by the foregoing table, expressed as percentages of the average leakage of the down-stream primary holes at the same depth appears as follows: At 10 ft., 20%; at 20 ft., 25.3%; at 30 ft., 21.7%; at 40 ft., 45.1%; at 50 ft., 75.2 per cent.

Back-Flow.

More strange, possibly, than the persistence with which the leakage continued, in spite of all the cement forced in, was the fact that with nearly all these holes, as with those put down in 1910, a very

DOWN-STREAM PRIMARY

Hole No.	637 a	638 a	639 a	640 a	641 a	642 a	643 a	644 a	645 a	646 a	647 a	648 a	649 a	650 a	Average
Test Date	9/8'11	9/29'11	9/15'11	8/26'11	8/22'11	8/29'11	9/1'11	8/19'11	8/30'11	8/24'11	8/30'11	8/23'11	8/29'11	8/20'11	
Gall's per Min	8	10	5	5	127	36.9	58.8	145	89.5	45.5	30	30	35.5	35.5	47.3
Grout Date	9/10'11	9/29'11	9/20'11	8/26'11	8/23'11	8/29'11	9/2'11	8/19'11	8/30'11	8/24'11	8/30'11	8/23'11	8/29'11	8/21'11	
Bbls. Cement	2	1.5	1.25	27	18.75	28	5	6.5	6.25	5	8.75	1.25	1	5	8.375

UP-STREAM PRIMARY

Hole No.	438 a	439 a	440 a	441 a	442 a	443 a	444 a	445 a	446 a	448 a	449 a	450 a	450 b	Average
Test Date	10/2'11	9/29'11	9/2'11	9/7'11	9/4'11	8/25'11	9/6'11	9/3'11	9/7'11	9/3'11	9/5'11	9/1'11	9/23'11	
Gall's per Min	36.8	29	11.6	31	40.2	35.5	44.5	75	137	102	135	16.8	45.5	51.0
Grout Date	10/2'11	9/29'11	9/2'11	9/7'11	9/4'11	8/25'11	9/6'11	9/4'11	9/9'11	9/3'11	9/6'11	9/1'11	9/23'11	
Bbls. Cement	1.75	1.75	22	2.75	3	3	2.5	6.25	3.75	10	4.25	16.25	2	6.09

FIRST PROVING

Hole No.	839 b	840 a	841 a	842 a	843 a	843 b	844 b	845 b	847 e	848 e	850 a'	Average
Test Date	10/7'11	10/6'11	9/14'11	9/11'11	9/22'11	9/27'11	9/15'11	8/12'11	9/6'11	9/12'11	9/11'11	
Gall's per Min	4.7	41	26	45.5	135	10.9	89	135	59	68.7	121.5	67.0
Grout Date	10/7'11	10/6'11	9/20'11	9/11'11	9/23'11	9/27'11	9/20'11	9/13'11	9/19'11	9/13'11	9/11'11	
Bbls. Cement	1.5	1.75	2	2	2.5	1.75	2	1	2.5	2.5	2	1.95

SECOND PROVING

Hole No.	839 a	840 b	841 b	842 b	843 e	844 e	845 e	846 e	847 b	848 a	849 a	Average
Test Date	10/5'11	10/3'11	9/25'11	9/30'11	10/6'11	10/3'11	9/24'11	9/22'11	9/27'11	10/4'11	10/4'11	
Gall's per Min	27	39.1	25.4	20.1	33.5	29	15.6	78.5	18	30.5	32	31.7
Grout Date	10/5'11	10/3'11	9/25'11	9/30'11	10/6'11	10/3'11	9/24'11	9/22'11	9/27'11	10/4'11	10/4'11	
Bbls. Cement	1.5	1.75	1.75	2	1.75	1.75	1.5	1.5	1.75	2	3	1.84

FINAL PROVING

Hole No.	837 b	838 e	839 e	840 e	841 e	842 a'	842 e	843 a'	844 a
Test Date	10/11'11	10/13'11	10/12'11	10/12'11	10/11'11	10/13'11	10/11'11	10/13'11	10/10'11
Gall's per Min	5.5	8.5	22.3	37.5	33	45.5	46.5	30.7	60.5
Grout Date	10/13'11	10/15'11	10/15'11	10/15'11	10/15'11	10/15'11	10/15'11	10/16'11	10/15'11
Bbls. Cement	2	1.25	1.25	1.25	3	2.75	1.5	1	1.75
Hole No.	845 a	845 b'	846 a'	847 a	848 b	848 b'	849 b	850 e	Average
Test Date	10/10'11	10/14'11	10/10'11	10/10'11	10/11'11	10/15'11	10/11'11	10/11'11	
Gall's per Min	35.5	34	32.9	33	9.5	73	76	21.8	35.6
Grout Date	10/15'11	10/16'11	10/16'11	10/16'11	10/16'11	10/16'11	10/16'11	10/16'11	
Bbls. Cement	1	1.25	1.75	1.25	1.5	1.5	1.5	1	1.55

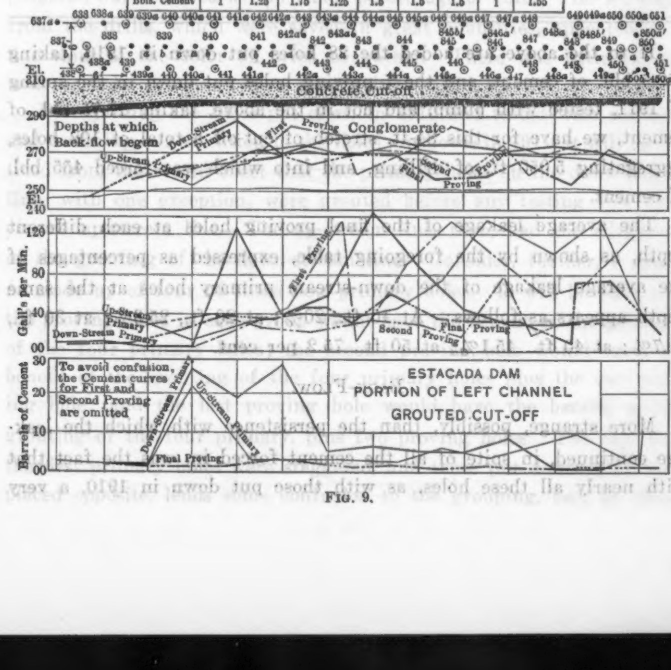


FIG. 9.

insignificant head produced a back-flow from the river. At this time the construction of the dam had completely blocked the right channel, so that operations were carried on in the left, behind coffer-dams which blocked off first one and then the other half of this channel, and from these resulted the head which produced the flow.

The average depth at which the back-flow commenced in the case of the downstream primary holes was 27.2 ft.; the upstream primary, 33.0 ft.; the first proving, 37.5 ft.; the second proving, 32.5 ft.; and the final proving, 25.0 ft.

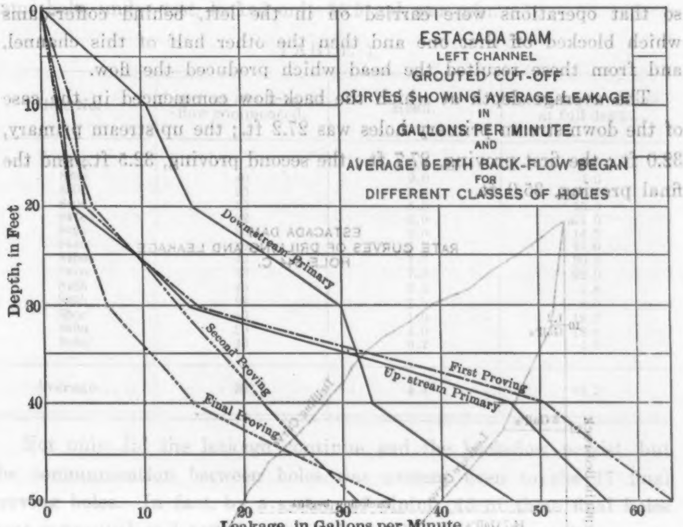


FIG. 10.

The depth below the bottom of the concrete cut-off at which this flow commenced, the head producing it, and the gallons per minute for 14 of the final proving holes, are given in Table 1.

In Table 1, under head, is given the vertical distance that the top of the casing is below the level of the water against the coffer-dam. This small head, averaging only 4.4 ft. for these 14 holes, produced a back-flow averaging 12.7 gal. per min., or one-third of the average leakage for the same holes when tested from a tank some 70 ft. above

insignificant head produced a back-flow from the river. At this time the construction of the dam had completely blocked the right channel, so that operations were carried on in the left, behind coffer-dams which blocked off first one and then the other half of this channel, and from these resulted the head which produced the flow.

The average depth at which the back-flow commenced in the case of the down-stream primary holes was 27.2 ft.; the up-stream primary, 32.0 ft.; the first proving, 27.7 ft.; the second proving, 32.5 ft., and the final proving, 35.0 ft.

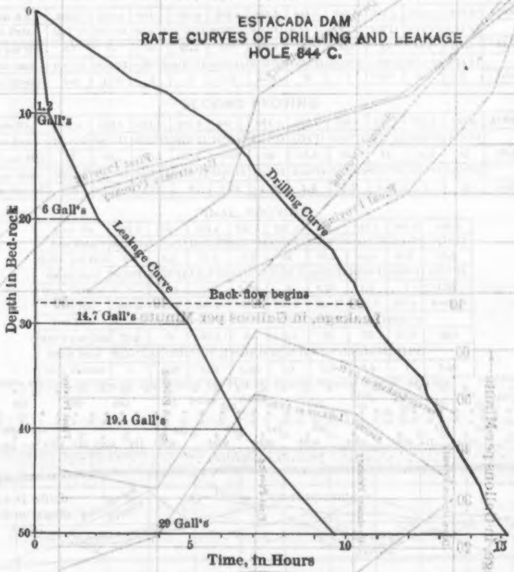


FIG. 11.

The depth below the bottom of the concrete cut-off at which this flow commenced, the head producing it, and the gallons per minute for 14 of the final proving holes, are given in Table 1.

In Table 1, under head, is given the vertical distance that the top of the casing is below the level of the water against the coffer-dam. This small head, averaging only 4.4 ft. for these 14 holes, produced a back-flow averaging 12.7 gal. per min., or one-third of the average leakage for the same holes when tested from a tank some 70 ft. above

the level of the water against the coffer-dam. In one hole, 845*a*, the back-flow, under a head of 7.7 ft., was 30 gal. per min., though the same hole, under test, leaked only 35.5 gal. per min.

TABLE 1.

Hole No.	Depth at which back-flow commenced.	Head.	Gallons per minute at full depth.
837*b*	..	....	0.0
838*c*	40	2.0	4.0
839*c*	35	5.6	13.0
840*c*	35	5.8	15.0
841*c*	32	2.9	11.0
842*c*	30	4.0	14.5
844*a*	30	6.7	17.0
845*a*	42	7.7	30.0
847*a*	33	7.2	22.0
848*b*	46	6.6	1.8
849*b*	28	1.0	8.0
850*c*	40	1.0	19.0
843*a'*	30	4.0	18.0
846*a'*	42	6.7	4.0
Average......	33	4.4	12.7

Not only did the leakage continue and the back-flow persist, but the communication between holes was present even to the 17 final proving holes. In fact, by a system of piping, 13 of these final holes were connected and tested at one time. This, as well as other combination or multiple tests at other points of the cut-off, showed much less average leakage per hole than the average of the same holes tested singly. However, the fact that 17 holes put down along a stretch of cut-off 84 ft. long, in which 84 holes before them had taken 427.5 bbl. of cement, could communicate so freely that pressure from a tank on the crest of the dam applied to any of them would cause water to flow from the other 16 was poorly offset by the fact that 13 of these same holes when tested in combination showed an average leakage of 17.5 gal. per min. as against an average of 34.4 gal. when tested individually. This reduced leakage in the cases of the combination tests was interpreted as indicating that several holes intercepted the same seam or seams.

Grout in the Cores.

At one point, where a trench was excavated subsequent to the grouting, the cement could be traced as a white thread completely filling a seam ½ in. wide for 38 ft. This, of course, was only a few

feet beneath the surface. At greater depths the only evidence of the diffusion, aside from the decreased leakage in testing, would be the cement brought up as core or in the cement flakes in the wash from the drills. The grayish color of the wash, indicating that the drill was penetrating grout, was of much more frequent occurrence than the finding of solid grout in the cores. Of course, where the drilling of the proving holes followed in a few days or weeks the drilling of the primary holes, it was to be anticipated that the cement would be ground up by the attrition of the bits, the shot, and the harder particles of rock. After a lapse of several months, cement core might be obtained, but even then it would be best to use a double-tube core barrel. However, all cores were carefully inspected for any traces of cement, and, though some *bona fide* samples were found, they were very rare.

The words *"bona fide"* are used advisedly. At one time, owing to the poor showing being made by the grouting, there was serious thought of discontinuing it altogether. At this juncture, as the writer discovered later, at least one driller, moved by a commendable desire to see his job continue, joined short pieces of core with cement, which sample, deposited in the core box, was later seized on and carried up to the office as conclusive evidence of the diffusion of the grout.

Tests After Completion of Dam.

For a period of nearly 2 weeks after the pond had been allowed to fill, to admit of completing the excavation of the tail-race, the sluice-gates would be opened at night and closed during the day until such time as the water was nearing the crest. This, save for pools and pot-holes, left the left channel dry for several hours each day, and during these intervals careful daily inspections were made to detect any springs resulting from the 80 to 90-ft. depth of water above the dam. None was found, and the only seepage of any kind was very small in quantity, coming from well up on the left slope. The dam was completed at the time the fall rains set in, and this very insignificant seepage was believed to be due to the filling of the swales and depressions which occurred at intervals in the nearly level bench topping the left bank at the site of the dam.

When the water was flowing over the crest, a core drill was taken inside the dam and two holes put down. The first of these was next

FIG. 12.—VIEW FROM LEFT BANK. ISLAND BEING CUT TO SLOPE OF DAM.

FIG. 13.—VIEW FROM LEFT BANK. DECK BEING EXTENDED TO INCORPORATE ISLAND IN DAM.

FIG. 12.—VIEW FROM LEFT BANK. ISLAND BEING CUT TO SLOPE OF DAM.

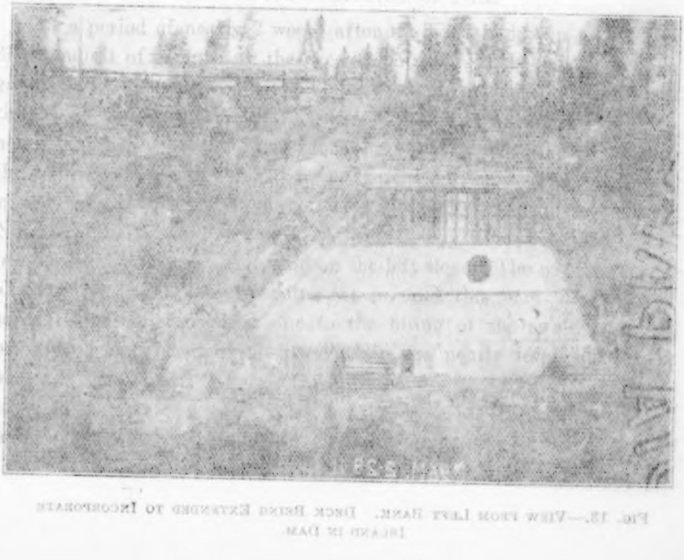


FIG. 13.—VIEW FROM LEFT BANK. DECK BEING EXTENDED TO INCORPORATE ISLAND IN DAM.

the left channel on what had originally been the island, and at a point 35 ft. down stream from the lower side of the grouted cut-off. Table 2, which is an abridgment of one appearing in Mr. Fisher's report to the owners of the plant, shows that water commenced to flow when the drill had penetrated the rock to a depth of 28.5 ft.

TABLE 2.—HOLE No. 1 000.

Elevation, top of casing.........................328.2
" of bottom of rock at hole..............325.0
" bottom of concrete cut-off opposite hole..306
" grouted cut-off opposite hole............248

Elevation, bottom of hole.	Elevation, water in pond.	Flow over top of casing, in gallons per minute.
296.5	375	First trace of flow.
289.5	376	0.9
280.5	387	2.2
270.5	387	5.4
260.5	387	5.4
249.0	387	6.6
245.5	387	7.5

The head, or difference of level, between the top of the casing and the water in the pond, with the exception of the first two determinations, is constant, and amounts to 59 ft.

A few days later, when the water in the pond stood at Elevation 388.4, the casing was extended upward with 1½-in. pipe, great care being taken to have all joints tight. When the water finally came to rest it was found to be at Elevation 364.2 or 24.2 ft. lower than the water passing over the crest above.

The other hole was put down directly in the left channel, and opposite that point in which the leakage had been most serious and the rock most refractive to tightening by the grouting process. This hole, No. 1 001, was opposite Hole No. 644*a* and 30 ft. down stream from the lower line of the cut-off. The first trace of flow from this hole was at a depth of 22.9 ft. in the rock.

In this hole the casing was extended and the height to which the water would rise was determined at intervals as the drilling progressed.

In this case the difference between the top of the casing, the point at which the flow was measured, and the elevation of the pond was nearly constant, and amounted to 79.5 ft.

TABLE 3.—HOLE No. 1 001.

Elevation top of casing............................315.1
" of rock at hole.............................308
" bottom of concrete cut-off opposite.......302
" " " grouted " " 248

Elevation, bottom of hole.	Elevation, water in pond.	Flow, in gallons per minute.	Elevation at which water came to rest.
285.1	....	First trace of flow.	315.1
283.1	387.5	0.7	368.0
281.6	387.1	16.9	369.4
270.1	387.8	26.0	369.6
255.1	387.5	32.9	369.7
250.1	387.1	46.0	373.1
247.1	387.1	53.8	374.0

In this hole, when the bottom was at Elevation 282.6, or 25.4 ft. in bed-rock, the drill suddenly dropped 2 in. and the water came up in greatly increased force and volume.

TABLE 4.—COST DATA.

QUANTITIES:		
Total drilled, 555 holes........................		34 038 lin. ft.
Average per drill per 10-hour shift........		13.2 " "
Average shot per drill per shift............		1.1 lb.
Primary or outside holes, grouted..........	375 taking	1 526.50 bbl. cement.
Proving or middle holes, grouted..........	160 "	275.25 "
Tight holes filled............................	12 "	15.00 "
Holes lost....................................	8 "	.... "
Setting casings, etc..........................	.... "	125.25 "
Total..	555 taking	1 942.00 bbl. cement.
Most cement in any one hole..................50 bbl.		
Average for 535 holes taking grout.......... 3.37. bbl.		
COST:		
Labor, drilling................................	$19 842.60	$0.59 per lin. ft.
Labor, grouting..............................	6 285.32	0.18 "
Cement, at $2.20 per bbl. at cut-off........	4 272.40	0.12 "
Repairs, oil, waste, shot, etc...............	5 908.35	0.17 "
Depreciation on grouting plant, 50%........	5 116.70	0.15 "
Total direct...................................	$41 425.37	$1.21 per lin. ft.
Other charges were prorated as follows:		
General plant, camp, etc.....................	$15 304.63	$0.45 per lin. ft.
Coffer-dams and pumping....................	5 121.63	0.15 "
Engineering and superintendence...........	6 414.91	0.19 "
Total cost.....................................	$68 266.54	$2.00 per lin. ft.

CONCLUSIONS.

In a piece of work of this character and magnitude, where much is experimental, nothing would be gained in attempting to present a

FIG. 14.—GENERAL VIEW, LOOKING DOWN STREAM. ISLAND CUT TO SLOPE OF DAM.

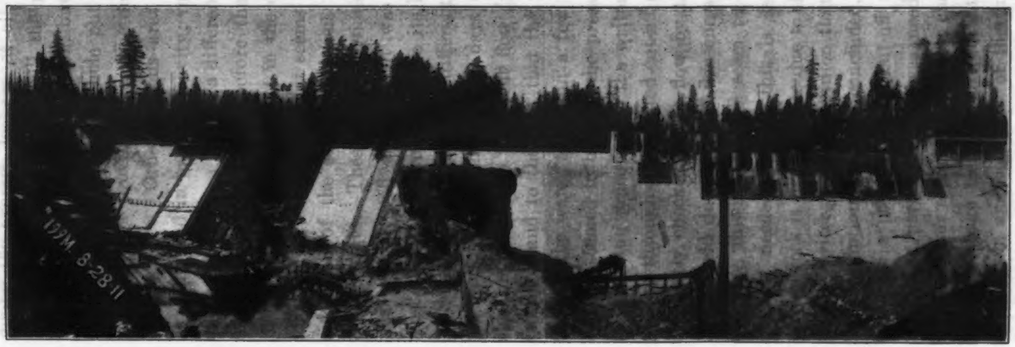


FIG. 15.—GENERAL VIEW, LOOKING DOWN STREAM, AFTER ISLAND HAD BEEN INCORPORATED IN DAM.

Fig. 14.—General View, Looking Down Stream. Island Cut to Slope of Dam.

Fig. 15.—General View, Looking Down Stream, After Island Had Been Incorporated in Dam.

review or an epitome of the work as a whole, such, for example, as giving the average leakage of all the primary holes and contrasting this with the average for all the proving holes. Such a marshaling would impart, not information, but misinformation. One of the reasons for this is that any hole put down near one already grouted is in a sense a proving hole, whether in the middle or in an outside line of holes. Other reasons are: that the testing was done in part by pump and in part by gravity; in part from a tank at one elevation and in part from a tank at a different elevation; in part through casings set in the conglomerate before the cut-off trench was concreted and in part through casings set in the concrete with which the trench was filled. For these reasons, it seems best to rest the case with the consideration of the several portions of the work taken up in the preceding pages. These cover portions of the cut-off where the material was best as well as where it was poorest; where the grouting was most effective as well as where its benefits were least marked.

Several engineers from different parts of the country, who visited the work during the closing days of construction, attached considerable importance to the great flow from the two holes put down inside the dam, and to the fact that the pressure encountered was sufficient to raise the water to within a few feet of the crest of the dam. The opinion seemed to be that the pressure and flow proved the failure of the grouting process. To the writer, they mean merely that the water passing through the grouted cut-off at a depth of 25 to 30 ft. is stopped or greatly checked in its flow by the denseness of the rock in front, and is held down by the denseness of the rock above. The pressure and flow, in view of this, seem to indicate, rather, that the grouting was not needed, though this is assuming that the tightness of the surface rock is in no degree due to the grouting.

The writer is of the opinion that most engineers who have followed this description will have concluded, in so far as this job is concerned, that over a portion of the cut-off no grouting was needed, and that over those parts where it was needed it did little good. The initial tightness over a part of the right channel and island sections, as shown by the seepage test and pressure tests, and the great leakage, communication, and back-flow of the final holes in the left channel afford ample basis for such an opinion. However, calculating for a head of 80 ft. and assuming an efficiency of 50% for power delivered to

the customer, it is found that, at $33 per kw-year, a saving of 26 sec-ft. would pay 7% on the $41 000 representing the direct cost of the grouting. Whether or not this quantity of water has been saved, no one can say. Even had the first primary holes shown very free leakage and the final proving holes tested absolutely tight, it would not have been known, so different are the conditions when water is forced through a casing into a hole 50 ft. in the rock from those when the pond is filled and there is no such penetration to the interior rock.

In rock having continuous seams, the grouting process would probably be much more effective than in the "volcanic débris" of the Clackamas Canyon. In this, though the grout will fill the larger cavities and interstices, it cannot be called a success, in so far as its being an agent for providing means for actually changing the structural character of the formation, as was hoped when beginning the work.

The knowledge gained by the experience at Estacada may be summed up as follows:

(1) Do all drilling, testing, and grouting through casings set in the concrete cut-off.

(2) Do all testing from elevated tanks, and not by pump.

(3) Test and grout each hole as soon as drilled, and for a few days thereafter keep the drills away from the probable zone of diffusion.

(4) In grouting, especially at high pressures, it is best to close the valve before the tank is entirely empty, as the air following the grout into the hole is apt to make trouble.

(5) Begin with a comparatively thin grouting mixture and, if taken freely, thicken until each succeeding batch requires either an increased time for discharging or an increased pressure. To force charge after charge of thin grout into a hole probably means in a great measure the wasting of cement.

The writer's opinion, now, is that either a single row of holes with close spacing or two rows of holes very close together in an up-and-down-stream direction, with casings staggered, is preferable to the triple line used at Estacada. At the Lahontan Dam, of the Truckee-Carson Project,* two rows of casings were put down, the distance be-

* *Engineering Record*, March 29th, 1913, p. 340; and *Engineering News*, April 3d, 1913, p. 647.

tween the rows was only 2 ft., and the distance between the holes of each row, 3 ft. As the casings in the two rows were staggered, it virtually amounted to a casing every 18 in. Over one section of this work, every fourth hole in the up-stream line of casings was first drilled, tested, and grouted. When the end of the cut-off was reached, the drills were returned to the beginning and the middle hole in each space was drilled, tested, and grouted, after which the drills were returned a third and a fourth time, drilling the remaining holes. This made every hole after the first, a proving hole, and as practical tightness was secured with the drilling of the up-stream line, the casings of the down-stream line were not needed, and were drilled through in only four or five instances as final proving holes.

The experience at Estacada indicates that the grouting should in no case be relied on in lieu of the usual concrete cut-off. There are two reasons for this:

(1) The efficiency of grout as a curtain-wall cannot be foretold.

(2) The proper diffusion of the grout can be secured only when the concrete of the cut-off closes the surface seams and confines the pressure to a depth at which it may be effective in tightening the underlying material.

Where very porous material is encountered below the practicable limit of depth for a concrete cut-off which, as in the case of the left channel at Estacada, proves refractive to the tightening by the grouting of isolated holes, the desired end might be secured by drilling a great number of holes close together and by springing the rock with small charges of powder from the bottom up to the top of the water-bearing stratum, thus shattering and loosening up the intervening material so that the grout would then form an impervious barrier.

As already stated, the pressure testing, though affording one means of measuring the effect of the grout, by no means proves it to be necessary.

In the case of a storage dam, the best plan would seem to be to set the casings in an offset cut-off just up-stream from the heel of the dam, as was done along part of the dam at Estacada. Through these casings a hole could be put down every 12 to 16 ft. to make sure that there were no large crevices. These casings should be tested and grouted and the need for further drilling and grouting left to be determined

by the filling of the pond. If the leakage were excessive and the seams did not tighten by the natural silting processes, the pond, at time of minimum storage, could be entirely emptied and the grouting continued. In this case it would be necessary to grout only at those points where seepage showed it to be needed, and, if tightness were secured, the engineer would have absolute proof of the efficiency of the process. Of course, with an earth dam like that at Lahontan, already mentioned, this method is impossible, and grouting must precede the construction of the dam proper.

In the case of a power dam, the same plan might be followed, and drilling and grouting done by drawing off the water at the time of minimum flow. Whether to do this or to carry the grouting along with the building of the dam, as was done at Estacada, is a problem the solution of which depends on what the minimum flow power is worth and whether the load might, without any great inconvenience, be transferred to other plants for a short time.

The consulting engineers on this work were Messrs. Sellers and Rippey, of Philadelphia. They were represented at various times and in various capacities on this and other projects of the Company, including the proposed 150-ft. dam on the Clackamas River, by Messrs. William H. Cushman, H. V. Schreiber, and S. C. Hulse, Members, Am. Soc. C. E., and W. L. Fitzgerald, Assoc. M. Am. Soc. C. E.

A. Gardner, Assoc. M. Am. Soc. C. E., represented the Ambursen Hydraulic Construction Company, of Boston, owners of the hollow-dam patents under which the dam was built, and L. I. Fletcher, Assoc. Am. Soc. C. E., was Superintendent for the contractors, the Puget Sound Bridge and Dredging Company.

Mr. T. W. Sullivan is Chief Hydraulic Engineer of the Portland Railway, Light and Power Company, owners of the plant, and Mr. F. R. Fisher was the Resident Engineer on the dam. The writer was Assistant Engineer of the same Company, and was connected with the work from the time of making the survey to the delivery of power.

DISCUSSION

Mr. Rippey.

S. HOWARD RIPPEY,* M. AM. SOC. C. E. (by letter).—The deep interest in the subject of grouting dam foundations, as shown by numerous private inquiries from engineers in America and abroad concerning the Clackamas River experiments, renders it certain that this paper will be generally appreciated by the Profession, and it is to be hoped will develop a broad discussion of the general subject of dam foundation treatment. Although this subject is always of interest to those engaged in hydraulic work, its seriousness is from time to time given added significance by such unfortunate occurrences as the failures of the Austin and Stony River Dams, as well as of other less conspicuous structures, showing that perfectly well designed and constructed concrete dams of any type may be wrecked as a result of inadequate preparation of their foundations.

The detailed information given by Mr. Rands concerning the practical execution of the grouting programme at Estacada should be of value to others facing similar problems, but the writer fears that some of the conclusions reached by the author and the personal opinions expressed, may tend to indicate to others undue difficulties and limitations in the general applicability of the grouting method to dam problems, and to that extent discourage the use of a method which in many instances may represent the only economically practicable solution of a development project.

In order, therefore, that the results obtained at Estacada, and the conclusions drawn therefrom by Mr. Rands, may be properly appraised by others interested in the treatment of deficient foundations, it is proper to outline, as briefly as possible, sufficient of the history of that project to indicate the unfavorable character of the governing conditions under which the work was executed.

As indicated by Mr. Rands, the use of grout at Estacada was prescribed by the writer, following thorough study of the foundation requirements at the site of a proposed large reservoir and power dam on Clackamas River, some 5 or 6 miles above Estacada (known as the Upper "Clackamas" or "Second Clackamas" project). At this site a mass concrete dam about 150 ft. high was found to be desirable as part of the most efficient plan of power development and flow equalization, but the character of the foundations (similar to those found later at Estacada), was such that the writer felt that the construction of such a dam on them would not be in accordance with any known precedent, good judgment, or the dictates of good practice, unless the objections could be overcome. It thus became necessary to decide whether to modify the development scheme so as to avoid the necessity for a high

* Germantown, Pa.

Mr. Rippey.

dam (as, for instance, by a low dam and a long flume to create the desired head), or undertake to improve the character of the foundations so as to justify the execution of the more efficient and satisfactory plan of development.

At this juncture the writer decided that the use of cement grout, injected under pressure, offered a possible solution of the problem of foundation treatment, and probably represented the only practicable method of treating this remarkably heterogeneous formation, so as to justify the construction of a high masonry dam on it. Test-pit and core-drill investigations in progress indicated the absolute need for novel measures in order to render the foundations fit for the proposed construction, and the formations encountered were such as to make the writer believe that the grouting method could be applied successfully.

In order to acquire a more comprehensive idea of the possible range of geological conditions to be encountered than was feasible through the local borings in progress and by superficial examination of the canyon in this locality, the writer, fortunately, was able at this time to secure a very thorough geological report from Professor J. S. Diller, of the U. S. Geological Survey, who made a personal examination and study of the canyon for the purpose. In view of the questions raised by Mr. Rands, the following abstract of Professor Diller's report is considered pertinent to the discussion:

"The rocks of the canyon walls are of four forms, volcanic breccias, lava sheets, volcanic dikes and terrace gravels. * * * The volcanic breccia (bed-rock) is made up of unassorted angular fragments of lava, andesite and basalt of various colors, ranging in size from dust particles and grains of sand to large rock fragments many feet in diameter. * * * Sheets of solid non-fragmental lava forming part of the bed-rock and outcropping on the slopes of the canyon occur within and between the great sheets of volcanic breccia. Some of the lava sheets are basalt * * * generally very porous. * * * Nearly vertical dikes of basalt cut up through the sheets of volcanic breccia and lava, and their outcrops on the surface have the direction of N. 65° W. approximately parallel to the general course of the canyon. * * * There is a set of parallel joints, the open cracks of which cut up through the volcanic breccias and sheets of lava about vertically in a direction approximately parallel to the course of the canyon. * * * Such joints may be of considerable extent and form important openings for the circulation of water. Well developed joint cracks of this system were not seen in the exposed bed-rock of the dam site, but they may be expected, and should be carefully looked for where the bed-rock is covered with soil or gravel. It is especially significant that the dikes are approximately parallel to these joint cracks and suggest that these joint cracks may extend to great depths. * * * From the nature of the volcanic breccia, which forms by far the greater part of the canyon walls, it is evident that the drill cores will differ from one another very much when compared. Where the drill goes

through a sheet of lava or a large solid fragment it will yield a good core, but where it penetrates the finer material (the volcanic ashes, in which the fragments of all sizes are imbedded), the core fails, the material is pulverized by the drill and washed away, and yet the extent of this material that is washed away is of the greatest importance, for it is the weakest element in the structure and the one which when saturated with water under pressure is most likely to become Engineers' 'soapstone'. Soapstone, properly so-called, does not occur in that region at all, but decomposed lava, volcanic ashes, and clay, all of which when saturated with water may become slippery and would be called 'soapstone' by Engineers, occur locally in the volcanic breccia. Large caverns and cavities, or pockets of loose earth and stones, are not to be expected in the volcanic breccia, but, owing to the manner of accumulation, there may be small openings, and the porosity of the rock is high. It is pervious to water, and for this reason similar material is used for making water coolers. The crushing strength of the volcanic breccia is, of course, small as compared with granite, limestone, and most other rocks, and this, taken in connection with its porosity and the possible existence of undiscovered joint cracks, seems to make a large reinforcement with concrete necessary, in order to furnish strength and prevent seepage as well as erosion. Mr. Rippey.

"The conditions that confront the engineer along the Clackamas River in the volcanic breccia plain region are very much the same as will be found all along the western foot of the Cascade Range from the Columbia River in Oregon to the Feather River in California—one of the most important water-power belts in the United States—and the successful solution of the problem it presents at one point will greatly facilitate the work elsewhere."

At this time only one of the core-drill holes had been subjected to water pressure as part of the original programme of studying the porosity of the foundations. In a report to his clients, covering instructions to the Field Engineer, Shirley C. Hulse, M. Am. Soc. C. E., the writer said at this time:

"The result of this test shows that 25 gal. per min., at a pressure of 175 lb., have been pumped into this hole for several consecutive hours, and, while coloring matter was used, no points of escape were noted and the behavior of the water surface in the surrounding holes has been so small and erratic as to be negligible. It has been evident since the first holes were drilled that we are dealing with a very porous and permeable foundation condition which will require special treatment to render it suitable for the construction proposed, and the purpose of the pressure tests outlined was to indicate the extent of permeability and the general lines of leakage, so that we might best plan to make the foundation of the dam impermeable. * * *

"In the Middle West, where a number of masonry structures are built on sand or earth foundations, it is customary to cover the river bottom for some distance up stream with a layer of clay and depend on the silt in the river closing up all the fissures and paths of seepage, but it would be very unwise to depend on this method in such a construction as we have under consideration. * * * It is necessary to

Mr. Rippey. solidify and render impermeable the formation immediately beneath our proposed dam before we are justified in building a dam at this site.

"It is evident that impermeable foundations are desirable: (a) to minimize the possibility of upward pressure under the base of the dam superstructure (this being a solid concrete dam); (b) to prevent percolation under the dam which might lead to sufficient erosion to involve undermining the structure; (c) to overcome any structural weakness due to the original geological formation, and properly provide a support for the superimposed load; and (d) to avoid waste of water (the chief power asset of the Company) from the reservoir, through, under, or around the dam, instead of through the turbines where it creates K. W. H. available for sale.

"It is * * * evident that we cannot afford to build the plant and find out afterward whether we can make the foundations impermeable, but that we should adopt a programme whereby we may hope absolutely to demonstrate our ability to render the foundations impermeable before the superstructures are built. * * * The general idea involves drilling a double line of holes under the heel of the dam and forcing into each hole grout of such consistency as to percolate through the permeable substructure and solidify it absolutely throughout the entire length of the superstructure, thus making the foundations absolutely solid and also providing the equivalent of an absolute cut-off wall. * * * Cement grout should be pumped into these holes and after time has been allowed for hardening, a third line of holes should be drilled midway between the first two lines and tested under water pressure. If water pressure is applied at or slightly above the hydrostatic pressure to which this rock would be subjected by the water in the reservoir after the completion of the dam, and it is found impossible to force any amount of water through the holes, we may feel reasonably certain that the cement grout has proven effective in making the entire foundation impermeable. * * * The test * * * may show a very large quantity of leakage, and this will simply mean that more holes would have to be drilled and pumped full of grout. * * * The programme to be followed will be necessarily indicated to a considerable extent by the experience obtained as the work progresses. * * * In order to avoid the uncertainties consequent upon an arbitrary determination of the spacing * * * of these grout holes, as well as of their depth, it appears expedient to select a convenient site where the cores indicate as difficult a formation to make tight as any on the job and to use this site as an experimental laboratory. * * * It seems probable, however, that this experiment may be made as part of the preparation of dam foundations, and the total cost of the work thus kept at the minimum. * * * We still regard it as essential that our ability to make the foundations tight be absolutely demonstrated before the work is started on the dam, and the experimental programme outlined provides for such demonstration."

This report included detailed descriptions and sketches unnecessary to reproduce here.

At about this time the company acquired from other interests the "River Mill" property, below Estacada, and for business reasons de-

cided to push a development there to early completion. The programme adopted by the owners of the properties provided that the detailed surveys, foundation tests, and preparation of detailed plans be made by the local organization and submitted to the writer, as Consulting Engineer, for his criticism and approval. The instructions issued specified: "This particularly applies to the foundation, as no chance or risk must be taken of locating a dam upon unsuitable foundations". Mr. Rippey.

The general character of development adopted was similar to that already planned for the higher-head, upper project, except that the hollow reinforced concrete type of dam, which the writer had selected for two earlier successful developments in New England, was adopted on his recommendation, and the grouted preparation of foundations on the general plan in progress at the upper site was prescribed.

The programme adopted in the interest of saving time, with the writer's hearty approval, was soon violated in many respects by the local organization (the personnel of which has since changed). The construction programme being followed was found to be such as the writer could not approve, and he, therefore, advised his clients that the modifications locally made in the foundation treatment were "beyond the limits of the precautions which should be taken in such an undertaking, and while no one can claim with any assurance that failure will result, I cannot conscientiously approve the programme adopted, nor assume any responsibility in connection with it."

The writer also stated, in another communication:

"The successful treatment of the foundations is dependent to a large extent upon the detailed methods adopted, but it is also dependent upon the spirit with which this work is carried out. Your organization has already expressed the opinion that detailed studies and grouting are unnecessary, and, further, that detailed programmes prepared in advance of actual exposure of foundations are useless.

"We therefore feel that, aside from the engineering questions involved, your organization has not a proper appreciation of the subject; and further, any programme that may be insisted upon which is not in accord with their ideas would not be carried out in the proper spirit."

The foundation work had been held up for several months because of the writer's inability to approve what was proposed and the unwillingness of the local organization to follow the programme ordered, when the President of the Company suggested that John F. Stevens, M. Am. Soc. C. E., late Chief Engineer of the Panama Canal, be asked to pass upon the questions at issue. As a result of the opinions expressed by Mr. Stevens* (who was practically an arbitrator selected

*By the time the situation was formulated for presentation to Mr. Stevens, the local organization had developed a programme for the drilling, testing, and grouting of foundations, coincidently with or subsequent to the construction of the dam and other superstructures, instead of prior thereto, and his formal report, therefore, related largely to a comparison of the methods of execution, rather than to

Mr. Rippey.

by the local parties), the owners of the property were confirmed in their support of the judgment of the writer, and insisted on the work being properly executed. So much time had been wasted, however, and the urgent need for power was so great, that it became necessary to carry along the grouting work in connection with actual construction of the superstructures in the best possible manner under the circumstances, and the conditions were not ideal for the development of the full possibilities of the use of grout.

The reversal of the local authorities and their reluctant acceptance of the ordered programme produced an attitude which resulted in unnecessary difficulties, obstructions, and personal controversies on the ground, finally leading to another change of Resident Engineers, for purely personal reasons. Under all the circumstances, it reflects great credit on the ability and adaptability of all the engineers engaged on the work that they were able to attain as good results as those described by Mr. Rands in the face of aggressive lack of sympathy displayed by the local organization toward the foundation programme.

As his clients loyally supported the writer's judgment in these matters, he regrets the necessity for discussing them in this connection, and at this late day. The Profession, however, is entitled to a knowledge of the actual conditions of the execution of the work, in order that other engineers may not be misguided by the results and by the opinions expressed. Moreover, as the writer's connection with the undertaking has been thus publicly recorded by Mr. Rands, the foregoing explanations properly become part of the record.

It must be understood that although the writer in 1910 declined to be considered responsible for results, in view of the conditions developed through the continued disobedience of governing instructions, he has no reason to question the safety of the final construction, which was

the accepted fundamental necessity for grouting the foundations. The following extracts from his report, however, indicate clearly his attitude on the larger question and his appreciation of the importance of foundation treatment:

"The limited number of test pits and borings which have been made shows clearly that, as far as concerns the area they covered, no well-defined, extremely hard masses of rock exist—that the rock is soft and shades away into clay, the latter appearing under superficial examination much like a half-burned brick. The whole formation, to the non-geologically scientific man, seems to be a mass of soft conglomerate. * * *

"The writer regards it as unfortunate that more test holes were not put down. * * * Enough testing, however, has been done to establish the fact that seams, or fissures, or cavities, do exist where certain of the holes were put down, and therefore it is only fair to assume that they exist where no testing was done; in fact, the only safe assumption is that they exist all over and through the formation, but to what sizes and shapes no one can say. There is no doubt in the mind of the writer that all such seams or cavities can be filled and sealed to exclude water by the proper introduction of cement grout. * * * If liquid cement under 200 lb. pressure cannot be forced into the material, the water under 35 lb. cannot find its way, and the writer concurs in such opinion. * * * Grout to be forced in by compressed air under 200 lb. pressure, the flow to be continuous from start to finish. This the writer believes to be the most practical and best method, * * * the writer believes that the material is, over the greater part of the area, porous enough so that under the heavy pressure the grout would form a practically continuous body, or curtain, from 12 to 15 ft. in thickness. * * * No living man can guarantee the absolute success of the grouting when it is placed."

nominally completed along the ordered lines, although under many handicaps. Mr. Rippey.

While the Estacada work was in progress, the company acquired other partly developed power properties, and the Upper Clackamas development was therefore postponed before any conclusive results had been reached in the grouting experiments at that site.

About 2 years ago, in discussing the Estacada Dam in relation to the original experiment, the writer stated:

"It will be evident that the question of uplift pressure under a dam of this type is practically negligible, and, as the static head created is only about 83 ft., the other risks incident to a permeable foundation become proportionately smaller. As an indication of the porosity of the foundation material, it was found that the average leakage through 43 holes each 50 ft. deep (individually tested) was at the rate of 82½ gal. per min. under an average pressure of only 17.7 lb. per sq. in. The problem here becomes that of reducing the permeability to reasonable limits to overcome any structural weaknesses, prevent objectionable erosion and limit waste of water, without insisting upon the degree of tightness which would be necessary if the upward pressure under the dam base were a factor. The work was very urgent and the construction programme was modified in numerous respects to meet the exigencies which developed, but the dam was completed in November, 1911, and the foundation treatment appears to have met the practical requirements, although it was not pursued to the extent required to entirely eliminate upward pressure (had a solid dam been adopted).

"The holes were not uniformly spaced throughout the foundation, being located as experimental tests and the related construction work indicated best under the circumstances, and while the dam has been successfully completed and the plant is in regular operation, it is probable that the foundation treatment could be more thoroughly and economically effected in another case if taken up vigorously during the preliminary stages of a development so as to be independent of construction complications. Moreover, the practical experience obtained should greatly facilitate adapting this method to another development. It will be quite obvious, however, that each site will have its own peculiarities, and radically different treatment may be required."

Although the writer thus recognized the incompleteness of the grouting results attained as a result of the conditions described, and the degree of subordination of the grout work to the construction programme which existed, it does not appear that the results were as negative as might be inferred from some of the personal conclusions reached by Mr. Rands. The very large quantity of cement introduced into the foundations necessarily represents an improvement in the formation to that extent, aside from any interpretation of pressure-test results, and, as Mr. Rands states, tests made after completion disclosed no leakage of water past the cut-off as the result of a hydrostatic head of from 80 to 90 ft. above the dam.

Mr. Rippey. Although the final official report of Mr. Frank R. Fisher, who was Resident Engineer in charge during the period of completion of the development, indicates clearly that the grout did not produce impermeability over certain limited areas, it shows very definitely the practical sufficiency, for the purpose, of the results attained, as indicated by the following quotations therefrom:

"The excavation of the trench for the supplementary cut-off wall, in the locality where grouting had previously been done, afforded an excellent opportunity for observing its effects, as many seams were exposed, and all of them proved to be well caulked with cement. None of any magnitude were observed, the largest ones averaging about $\frac{1}{2}$ in., and varying from that down to $\frac{1}{16}$ in., the smaller ones also having taken the grout freely.

"There was a total of 29 proving holes put down on this [the right channel] section, and, discarding the tests on two of them that represented excessive leakage that was only local, the average for the remaining 27 was 3.6 gal. per hole. In material of this kind this was not considered excessive, and it is reasonable to assume that for all practical purposes the cut-off is effective on the right channel section.

"*Conclusions.*—The final conclusions to be drawn * * * may be stated as follows:

"*1.*—It is not feasible to accomplish an absolutely impervious cut-off by this method, in foundation material of the character that exists in this region. The method apparently is effective in closing up seams and crevices of appreciable size, as all such exposed by the excavations made subsequent to the grouting were tightly caulked with cement.

"Where seepage occurs through a more or less porous formation, or through veins of compact broken rock mixed with sand or a certain kind of clay rock, very much seamed, but with the joint or cleavage faces in close contact, all of which have been observed to exist over very limited areas in this locality, the grout has but slight effect in benefiting the condition. Material of this character acts as a filter, the cement remaining in the hole, and it cannot be diffused throughout the surrounding mass.

"*2.*—While the grouting did not result in an impervious cut-off, the amount of possible seepage through the foundations was very much reduced thereby and it may be considered tight to the extent that erosion and appreciable waste of water would not occur.

"*3.*—The tests on the two holes put down inside of the dam, after the filling of the pond, indicate the presence of some upward pressure in the foundations. It is questionable, however, whether this could be prevented, even with an impervious cut-off 50 ft. in depth, for the indications are that the material in this particular locality is such that water would travel downward and pass under any depth of cut-off that it would be practicable to accomplish.

"*4.*—As it has been proven that the foundations cannot be grouted absolutely tight, it is obvious that but little good is accomplished in attempting to reduce still further what might be termed a reasonable rate of seepage by the introduction of additional holes."

The interest manifested in the Clackamas grouting experiments and the successful adoption of similar methods elsewhere clearly demonstrate the fundamental merit of this method of treating deficient foundations. Indeed, in some cases it appears to the writer to afford the only practicable means of rendering certain formations satisfactory for the construction of high solid masonry or concrete dams. The problem of eliminating upward pressure under the base of solid dams, the significance of which was emphasized in the discussion of a paper* by the late C. L. Harrison, M. Am. Soc. C. E., is peculiarly susceptible of solution by the grouting method in certain formations. In the course of the discussion on that paper, Arthur P. Davis, M. Am. Soc. C. E., said: Mr. Rippey.

"Recent experience has shown the feasibility and efficacy, in some cases, of closing the crevices in the foundations wholly or partly by grouting them under pressure. This was accomplished successfully at moderate cost on the Ashokan Dam, and on several others of recent construction. The most striking instance of this kind which has come to the writer's attention is the Clackamas Dam, in Oregon, which was built on a foundation of semi-indurated volcanic ash, which was checkered in all directions by innumerable fissures, and, furthermore, was so soft that percolation was likely to cause destructive erosion. A triple line of holes was grouted along the up-stream toe of this dam, and recent information is that, since the dam has been in use, no perceptible percolation has taken place.

"The effect of such grouting is not easy to foretell, and, like all other underground conditions, must be estimated with extreme caution."

Incidentally, it was the failure of the local organization to recognize the necessity for this extreme caution, as emphasized by so experienced an authority as the Chief Engineer of the U. S. Reclamation Service and insisted on by the writer, in connection with the Clackamas work, which led to the unfortunate lack of sympathetic co-operation at Estacada.

A private letter from certain foreign engineers who had inquired about the Clackamas grouting experiments, from which the writer does not feel at liberty to quote directly without permission, reported the complete success of grouting to create an impermeable mass of a defective formation through which water freely escaped from a reservoir. It appears that the greatest success was secured by using a very dilute grout mixture, starting with quite low pressure, and finishing with pumped injection, whereby a certain amount of shock was applied which tended to overcome any temporary obstruction. It was demonstrated that the grout, which set well, traveled more than 100 ft., in plan, and the results appeared satisfactory in the highest degree.

* "Provisions for Uplift and Ice Pressure in Designing Masonry Dams," *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 142.

Mr. Rippey. In a recent article in a technical journal,* D. W. Cole, M. Am. Soc. C. E., Project Engineer, U. S. Reclamation Service, describes the application of the grouting method to the foundations of the Lahontan Dam of the Truckee-Carson Project. The writer understands that Mr. Rands was in direct charge of the grouting work (following the completion of the Estacada work), and Mr. Cole describes the results as "excellent", evidently considering, in advance of the completion and filling of the reservoir, that they were successful in accomplishing their purpose. He refers to the fact that it was proven that the grout hardened in place and gives evidence of the effectual sealing of fissures and seams and the solidification of the foundations adjoining the grouted holes.

Another article, in the same journal,† describes the grouting work done on the foundations of the Olive Bridge Dam of the Ashokan Reservoir, part of the Catskill water supply project for New York City. Two definite planes of slight seepage, 40 and 60 ft., respectively, below the creek bed, were found. It appears that 1 439 cu. ft. of thick grout were injected (under comparatively low pressure), of which it was estimated that 1 100 cu. ft. entered the seams. No proving tests seem to have been made.

Ample evidence exists to demonstrate the broad possibilities of the grouting method in its application to the solidification of defective foundations, and it remains for the engineer in each case to adapt it to local requirements. Of course, no general engineering method or formula can eliminate the need for experienced judgment in its detailed application, and, in work of this character, sympathetic co-operation is necessary, down to the humblest participant in its execution.

The writer believes that the grouting method affords many economical possibilities in water conservation and power development. It may permit the creation of an operating head entirely by a high dam, instead of by a low dam and a long, and perhaps leaky, flume; it may permit the selection of a site for a dam where the topography is favorable but where the natural foundation is impossible or not as satisfactory as at some poorer power location on the stream; and in many ways may be developed to render reservoir and power problems more flexible.

Even where the security of the construction may not be in question, the prevention of waste by percolation may amply justify the grout treatment to secure impermeability. At the Upper Clackamas, for instance, it was estimated that a saving of only 26 sec-ft. would justify an expenditure of $80 000, aside from any of the collateral

* *Engineering Record,* March 29th, 1913.

† *Engineering Record,* April 8th, 1911.

advantages (there real necessities) of preventing the flow of water under the dam. Mr. Rippey.

It is difficult to imagine any alternative method of foundation treatment in which the range of possibilities is as great as in grouting.

It will be observed that an essential feature of the grouting programme proposed by the writer is the ability to demonstrate by test, before building the superstructure, that the solidification of the porous material to a satisfactory degree of impermeability has been effected. The holes drilled by rotary drills for securing core specimens, and thus studying the foundations, were utilized later for the hydrostatic pressure testing, for washing out loose and soluble material, and for the introduction of grout under pressure. In the peculiar and exceedingly variable material encountered, it was found that in some cases communication existed between holes as far apart as 70 ft., and in other cases grout could not be made to permeate the material sufficiently unless the holes were very close together. The result was that a greater number of grout holes were required than was tentatively assumed without any experimental knowledge whatever, and a greater number than were required for core studies alone. This fact suggests that, in other applications of this method to large undertakings, only sufficient core holes be drilled by rotary drills to permit proper studies of the geological formation and general foundation structure, and that the remaining holes be sunk purely for grouting and testing purposes by apparatus which will perform the work more rapidly and economically.

The nature of the operation involved in securing cores is such as to necessitate slow progress and relatively high cost per foot of drilling, and when the writer reached a point on the original work where he was not specially interested in securing additional core samples, and where the results showed the need for closer spacing of grout holes than was thought from the general appearance of the rock and its permeability to clear water would be necessary, he considered the use of drills which would perform the remainder of the work more expeditiously and at less cost than was possible with rotary drills made primarily for securing cores.

The average rate of progress with the core drills was about 13 ft. per 10-hour shift, and it was evident that if the further work was confined to plain drilling it could be done much more rapidly. It appeared that the future drilling might best be done with percussion drills with guided rods arranged so as to drill holes 6 in. in diameter and about 50 ft. deep, approximately plumb, at a high rate of speed. A number of drills were found on the market which could be mounted to accomplish this work, but there was great difference of opinion among the manufacturers as to the possible rate of drilling in this material, estimates running from 25 to 150 ft. per 10-hour shift. At

Mr. Rippey.

this juncture the development was postponed, for the business reasons stated, and the subject was not pursued further, but there is evidently opportunity for great saving along these lines.

The costs of drilling and grouting at the Estacada Dam, as given by Mr. Fisher in his final report, and by Mr. Rands, on a somewhat different basis, in his paper, and the cost of grouting given by Mr. Cole for the Lahontan Dam, all reduced to a unit basis, are given in Table 5.

TABLE 5.—COST OF DRILLING AND GROUTING AT ESTACADA AND LAHONTAN DAMS, PER LINEAR FOOT OF COMPLICATED WORK.

Labor and materials.	ESTACADA DAM.		LAHONTAN DAM.
	Fisher.	Rands.	Cole.
Labor, drilling	$0.58	$0.59	$0.93
Labor, grouting	0.18	0.18	0.29
Cement	0.12	0.12	0.31
Repairs and supplies	0.17	0.17	0.28
Plant	0.30	....	....
Plant depreciation	....	0.15	0.35
Power	0.05	....	0.03
Other items	....	....	0.94
	$1.40	$1.21	....
Salvage on plant, Credit	0.17	....	....
Direct cost	$1.23	$1.21	....
Total field cost	....	....	$3.08
General plant, etc.	0.32	0.45	0.12
Coffers and pumping	....	0.15	....
Engineering and superintendence	....	0.19	0.27
Clerical and office	....	....	0.10
Total cost per foot	$1.55	$2.00	$3.57

It is of interest to note that the cost of cement per foot of hole at Lahontan was more than 2½ times that at Estacada; examination of the data, however, indicates that cement cost about $2.77 per bbl. at Lahontan and only $2.20 at Estacada. The actual quantity of cement used per foot of drilled hole appears to have been as follows:

Estacada.—34 038 ft. of holes: 1 942 bbl. = 0.057 bbl. per ft.

Lahontan.—2 593 ft. of holes; 1 174 sacks, say, 294 bbl. = 0.113 bbl. per ft.

In his final Estacada report, Mr. Fisher says: "The holes put down with the diamond drills cost about one-third more per foot than with the shot drills."

The average rate of drilling at Estacada was 1.3 ft. per hour and at Lahontan about 0.75 ft. per hour.

The use of percussion drills for grout holes after all necessary cores are taken, as has been suggested, should materially reduce the total drilling cost on a large job, and should also reduce the time element, which is of prime importance in most river work. Mr. Rippey.

The writer has information concerning other successful grouting in dam foundations in a foreign development, but, unfortunately, is not at liberty to communicate it at this time. Although the method described has inherent possibilities possessed by no other system, cases may arise wherein the impregnation of an extended rock mass with grout is not necessary, but where a definite and absolute sub-surface cut-off is required. There are many difficulties and limitations incident to the construction of a cut-off wall in an excavated trench for this purpose, especially for work under a high head where the desired depth is great, and, in connection with the consideration of the grouting scheme, certain alternatives for such limited applications have been developed. No opportunity has occurred for testing these, but some suggestions concerning them appear to be pertinent to this discussion, and may lead to the further development of foundation treatment methods.

Fig. 16 illustrates the general features of a suggested cut-off; this plan contemplates drilling a single line of 6-in. holes on 9-in. centers, broaching the intervening webs so as to form a continuous slot, introducing interlocking steel piling to ensure a water-tight barrier to the passage of water, and firmly securing this steel curtain in place and backing it by concrete, in the manner shown. The introduction of this curtain would afford the assurance of a positive stop to the percolation of water, but would not provide for the solidification of the mass of the foundation so as to increase its bearing value. For this reason this method might be specially applicable to a stratified foundation where approximately horizontal seams or bedding planes containing clay or soft material constitute a source of danger only when subjected to erosion.

In considering the proposed method of constructing an effective cut-off barrier of this type, the details of execution remain to be studied. There appears to be no doubt that any one of several makes of percussion drills is capable of drilling 6-in. holes 50 ft. deep at relatively high rates of speed, but in ordinary work the question as to whether such holes are straight and truly plumb has been given little consideration. In the proposed scheme, the necessity for inter-connecting the drilled holes by broaching or blasting out the intervening webs to form a continuous slot, necessitates a certain degree of parallelism of the adjacent holes, and, as such drilling is without precedent for holes of this depth, the drill manufacturers have naturally been unable to make any guaranties as to performance in this respect, but there appears to be no doubt as to the possibility of working out this feature of the

Mr. Rippey.

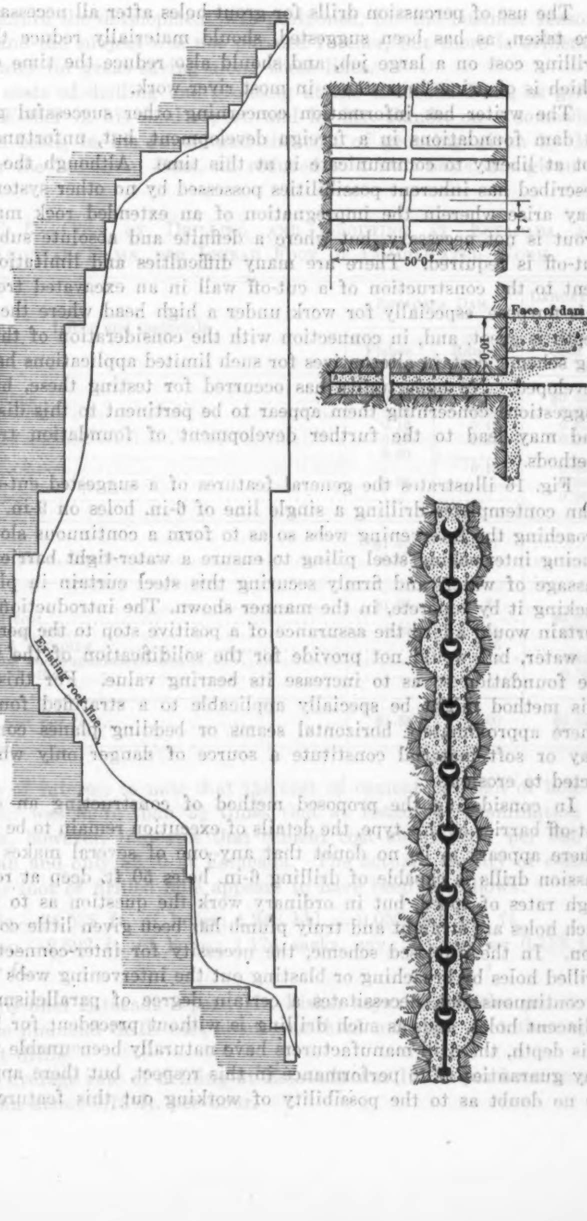


FIG. 16.

problem. Incidentally, the manufacturers have nothing to offer in the way of a machine for directly channeling such a slot as required, and the slot seems to be physically, as well as in the conception of method, the natural development of a series of holes drilled reasonably close together. Mr. Rippey.

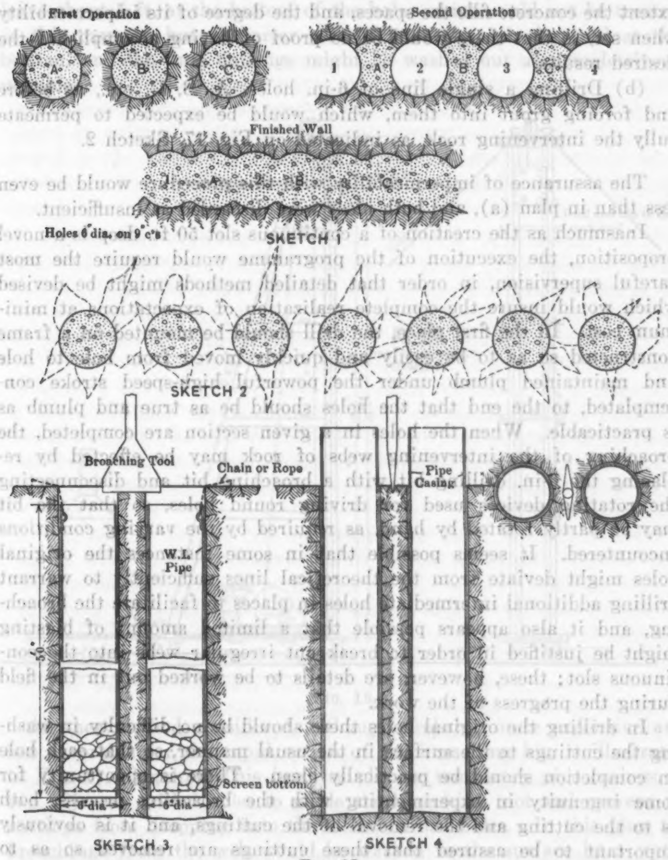


FIG. 17.

As alternatives to the plan proposed, wherein the steel curtain provides the absolute cut-off within the limits of its depth, two modified schemes have been considered:

(a) Drilling a row of 6-in. holes, A, B, C, etc., on 9-in. centers, as shown by Fig. 17, Sketch 1, filling these holes solid with cement mortar

Mr. Rippey. or grout, and, after setting, drilling intermediate holes, 1, 2, 3, etc., which in turn would be grouted, thus forming a continuous concrete wall of 6-in. maximum thickness, if the holes are all plumb and parallel. There would be no way of determining, however, to what extent the concrete fills the spaces, and the degree of its impermeability when set, so that there would be no proof of having accomplished the desired results.

(b) Drilling a single line of 6-in. holes, A, B, C, etc., as before and forcing grout into them, which would be expected to permeate fully the intervening rock, as indicated on Fig. 17, Sketch 2.

The assurance of impermeability with this procedure would be even less than in plan (a), and both ideas were discarded as insufficient.

Inasmuch as the creation of a continuous slot 50 ft. deep is a novel proposition, the execution of the programme would require the most careful supervision, in order that detailed methods might be devised which would insure the complete realization of expectations at minimum cost. In the first place, the drill should be mounted on a frame constructed so as to be easily and quickly moved from hole to hole and maintained plumb under the powerful high-speed stroke contemplated, to the end that the holes should be as true and plumb as is practicable. When the holes in a given section are completed, the broaching of the intervening webs of rock may be effected by replacing the 6-in. drilling bit with a broaching bit and disconnecting the rotating devices used for driving round holes, so that the bit may be partly rotated by hand, as required by the varying conditions encountered. It seems possible that in some instances the original holes might deviate from the theoretical lines sufficiently to warrant drilling additional intermediate holes in places to facilitate the broaching, and it also appears possible that a limited amount of blasting might be justified in order to break out irregular webs into the continuous slot; these, however, are details to be worked out in the field during the progress of the work.

In drilling the original holes there should be no difficulty in washing the cuttings to the surface in the usual manner, so that each hole on completion should be practically clean. There is opportunity for some ingenuity in experimenting with the broaching process, both as to the cutting and the removal of the cuttings, and it is obviously important to be assured that these cuttings are removed so as to interpose no obstacle to the introduction of piling or the complete filling of the slot by solid concrete. This cleaning out of the broachings may require some experimentation to secure the best results, but one method which suggests itself is illustrated in Fig. 17, Sketch 3. Two standard wrought-iron pipes, having an outside diameter of $5\frac{9}{16}$ in. and perhaps 5 ft. long, are indicated, with the lower ends covered

by a close-meshed wire netting which will pass water, but not cuttings. Mr. Rippey. These tubular buckets, supported by chains with their tops just below the lower limit of stroke of the broaching bit, would collect almost all the cuttings, being raised and emptied when full. This would leave about 5 ft. at the bottom of the holes which could not be treated in this manner, but a shorter pair of buckets might be used at the bottom, and the final cuttings might be washed out as completely as possible.

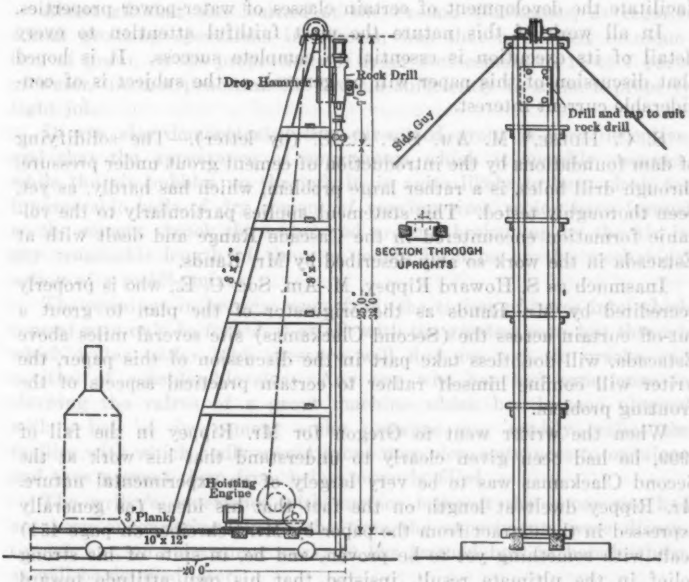


FIG. 18.

Another method considered, although probably more expensive and less favorable for the rapid action of the broaching bit, is illustrated in Fig. 17, Sketch 4. This involves placing temporary pipe casings in two adjacent drilled holes to guide the broaching tool and prevent its cuttings from dropping into the clean, drilled holes. In this plan the broachings from the web would be washed to the surface by a water jet in the same manner as in the drilled holes; a few trials should determine the most effective method.

As indicated, a limited amount of blasting in the slot may be found expedient, and there is also a possibility that a heavy rectangular tool might be used finally, in pile-driver fashion, to break out any remaining projections and establish the required width of continuous

Mr. Rippey. slot to the bottom. The general scheme of mounting the drill is shown by Fig. 18.

Although there has been no opportunity to develop the curtain cut-off, no insurmountable difficulties appear, and it should be entirely practicable to perfect apparatus for creating the deep slot required. This should assist materially in the solution of the problem of providing an effective cut-off in general practice; and convenient commercial apparatus of the character required might thus greatly facilitate the development of certain classes of water-power properties.

In all work of this nature the most faithful attention to every detail of its execution is essential to complete success. It is hoped that discussion of this paper will be general, as the subject is of considerable current interest.

Mr. Hulse. S. C. Hulse,* M. Am. Soc. C. E. (by letter).—The solidifying of dam foundations by the introduction of cement grout under pressure, through drill holes, is a rather large problem, which has hardly, as yet, been thoroughly tested. This statement applies particularly to the volcanic formation encountered in the Cascade Range and dealt with at Estacada in the work so ably described by Mr. Rands.

Inasmuch as S. Howard Rippey, M. Am. Soc. C. E., who is properly accredited by Mr. Rands as the originator of the plan to grout a cut-off curtain across the (Second Clackamas) site several miles above Estacada, will doubtless take part in the discussion of this paper, the writer will confine himself rather to certain practical aspects of the grouting problem.

When the writer went to Oregon for Mr. Rippey in the fall of 1909, he had been given clearly to understand that his work at the Second Clackamas was to be very largely of an experimental nature. Mr. Rippey dwelt at length on the fact that his ideas (as generally expressed in the extract from the paper by Mr. Schreiber on page 454) dealt with something yet to be proven, and he, in spite of his strong belief in the ultimate result, insisted that his own attitude toward the results of the experiments to be made, would be most conservative.

The experiments at the Second Clackamas were halted (by the postponement of the work for business reasons) before any positive results were obtained, as regards the actual efficiency of grouting. Somewhat more than 5 000 lin. ft. of core-drill holes were put down, and these ranged from 40 to 250 ft. in depth. Many and exhaustive pressure tests were made in these holes, and they ranged, from a few which were absolutely tight against 200 lb. pressure of water, to those which took water freely at almost no pressure. Often the holes were intercommunicating and sometimes the water showed on the surface of the adjoining territory—although this did not often

* Bedford, Pa.

occur. In many cases it was impossible to discover what became of the water that some of the looser holes took freely, although very great effort was made to trace it. Mr. Hulse.

After many delays and disappointments, a grouting machine was landed on the work, and experiments were begun, but, before these had proceeded to any very definite result, the machine was ordered to Estacada where things were going with such a rush as to overshadow the necessity for further investigation at the upper site.

About all that was learned at the Second Clackamas, as regards actual grouting operations in breccia, was that a hurried introduction of grout, in the proportions of from 1 of cement to 1½ to 7 parts of water, did not penetrate the breccia sufficiently to make a properly tight job.

It was also demonstrated that air-mixed grout is very deceptive, and that the appearance of smoothness which it speedily assumes, while the air bubbles up through it, is quite likely to be a cloak for innumerable balls of dry cement of varying sizes, which have formed as the cement struck the water and are not broken up by the air in any reasonable length of time, as would be done by the mechanical action of a paddle mixer.

The openings in breccia range from the tiniest of pores into which cement may only be forced, if at all, with the greatest care, but through which water under high pressure will find its way, to cracks and cavities of considerable size. Any one who has had experience in cleaning the valves of a grout machine which has become plugged with a ball of dry cement under pressure may readily realize the facility with which badly mixed grout may close entrances to openings and thus prevent them from being properly filled.

The writer's experience with surface leaks at the upper site had not been encouraging, up to the time the experiments were discontinued, but, on subsequent work, he has learned how to handle these, as will be detailed later.

To proceed to the conclusions arrived at on page 480:

(1) The advisability of a concrete cut-off wall under the foundation of a dam is open to serious question, and the writer has heard more than one able and experienced engineer declare against the practice. To put in such a cut-off, a trench must first be opened. If this is done by hand, in any material capable of bearing a great dam, the work is likely to be slow and costly. If it is done by blasting, and particularly in breccia, where the use of a channeler is said to be impracticable because of the nature of the material, there is great danger of doing almost irreparable damage to the surrounding material, and this is exactly what happened in the left channel at Estacada, where the breccia adjoining the heel-trench was shaken and cracked in all directions by the blasting. The writer believes that

Mr. Hulse. grouting, properly done, may be used as a satisfactory substitute for a concrete cut-off, and to the top of the foundation rock.

(2) The testing of drill holes is such a fascinating pursuit (and the writer speaks from experience) that it may easily be carried beyond the limits of practical usefulness. However, the tank method is quite preferable where much testing is to be done.

(3) This method of procedure might readily entail an almost prohibitive amount of shifting about, and, if carried to its logical conclusion, might even lead to a complete and wholly unwarranted interruption of the grout and drill work. More will be said about this in connection with Conclusion (5).

(4) This conclusion is very true, and emphasizes the desirability of a grouting machine which may be operated by water pressure instead of by air. The following of water into the hole, behind the grout, would not be objectionable, whereas the air is a nuisance, and has to be closely watched, in this respect. The writer has hopes of developing such a grouting machine.

(5) and (3) There are two sorts of grouting which may be accomplished: plugging, and sedimentation or silting. There are several ways of going after subterranean territory, through the medium of a drill hole, in the hope of making that territory impervious to the passage of water under pressure, and the writer wishes now to present the method which seems to him best suited to the penetration of every opening of sufficient size to admit cement particles in the form of grout. This method is practically irrespective of the nature of the rock to be grouted.

The first necessity is a reliable driller, who realizes his responsibilities and will enter into the spirit of things. Fortunately, such drillers are not the exception, although one does occasionally meet with an experience such as has been mentioned by Mr. Rands in connection with his search for cement in the cores.

Casings should be set beyond the possibility of their coming loose. If this is impossible—as is sometimes the case in soft breccia—a mat of concrete, preferably to be incorporated later in the main structure, may be laid on top of and well bonded to the rock surface, at or near the heel line, and the casings set in this mat. The mat should be as narrow as possible, so that it will not close any more surface leaks than necessary.

If a core drill is used, the loss of drill water or the striking of a seam or crevice is the best indication of the time to test the hole. If a percussion drill is used, it may be well to test at stated depths—say, every 10 ft.—because the evidence of what is encountered by a percussion drill is not nearly so reliable as in the case of a core drill.

The point is that leaks should be grouted as soon as encountered. This is for the purpose of closing openings which would let the pressure

Mr. Hulse.

out of the lower portions of the hole (yet to be drilled) were they not closed. In the absence of special equipment for testing purposes, charges of air or water blown through the grouting machine will answer perfectly well to indicate how freely the hole may be expected to take grout, and to show up any leaks on the surface.

The first grout introduced should be quite thin—say, 1 part of cement to 30 parts of water—and it should be absolutely free from lumps of any sort. Pressure should be applied gradually, and the instant a surface leak shows, the pressure should be dropped to such a point that only water (perhaps colored with cement) flows from the leak. This is the starting point for a successful closure of that leak by a process of sedimentation, as contrasted with the possibility of plugging it immediately by using thick or lumpy grout. From this time forward it is a matter of patience and of judgment in the application of pressure and the thickening of the mixture, and a great deal of patience is worth a very little thickening, as judged by the final results. Cement is never wasted by this process of "slow grouting" as long as it does not appear at the surface, and a careful nursing of the pressure will deposit the cement underground where it is wanted. In time—sometimes very quickly—the leak fills, and, as the resistance increases, more pressure is applied, until finally the hole refuses at the maximum pressure available. An exploration of many surface leaks thus "slow grouted" under the writer's direction, has disclosed the fact that they had been tightly filled with cement, and so very tightly as to preclude any subsequent opening due to shrinkage, as would have been the case had they been merely filled with the thickest grout that could have been run through them.

There is nothing in this sort of treatment to preclude the almost immediate washing out of the drill hole (after refusal) and the continuance of the drilling to the next point where grouting may seem advisable. Holes grouted with thick grout will frequently take more grout, if re-tested within a few hours after the first refusal at maximum pressure, but it is very exceptional when one succeeds in making any impression whatsoever in re-testing a hole which has been "slow grouted" as just set forth. In the first case, the openings are plugged with a mixture which subsequently shrinks; in the second case, they are silted full so tightly as to preclude shrinkage.

As an illustration of the results to be gained by patience, the writer once grouted for eight continuous days and nights on a hole that took somewhat more than 200 tons of cement. For several days, thin grout was literally poured into that hole—much of it by gravity—and the grout was kept thin until the hole showed signs of closing up. Had Conclusion (5), as stated by the author, been followed, the operation would doubtless have ended much sooner, but the result (the hole, thus far, has proved to be tight against a head of 130 ft.) might have

Mr. Hulse. been quite different. Incidentally, of the more than 200 tons of cement used, perhaps a dozen sacks were wasted through the leaks which developed—surely not a high price to pay for placing where it was wanted the rest of the quantity used.

There is good evidence that Portland cement sets very slowly—if at all—after it has been introduced underground in this fashion, but, should the process of grouting necessarily entail an assumption of the necessity for the setting of the cement used? Rather should it not be looked on as an enforced closing of openings or passages by the silting or sedimentation of an insoluble material, introduced under an ultimate pressure greater than any which might subsequently tend to dislodge it? If the cement does set up, so much the better.

If the suggestion of the author, in connection with the grouting done at Estacada, that: "* * * so far as this job is concerned, that over a portion of the cut-off no grouting was needed, and that over those parts where it was needed it did little good", is to be sustained, what of the 1 920 bbl. of cement that were used? It is hardly to be assumed that all this cement went to close the cracks opened by the blasting of the heel-trench and to waste in dealing with surface leaks. That portion of the 1 920 bbl. which was not used for these two purposes must have gone somewhere in the foundation and have closed openings which would much better be closed than left open—else, why grout, or why bother about cut-offs? The action of the two test holes which were drilled inside the dam and flowed water, as set forth by the author, indicate obviously enough that the grout as applied did not close the breccia tightly, or, that the water passed under the curtain and then rose; and the writer is strongly inclined to the former view of the matter, but does not feel that the evidence in the case is to be viewed as damaging to the grouting system of tightening foundations. Rather, he believes that the Estacada work shows remarkable results in favor of the system. We have here, in the beginning, a case of a conservatively worked out scheme which is to be based on experiment and subject to experimental proof. Before anything much has been learned from the experiments, they are interrupted and a practical application is made, and under circumstances by no means favorable to a fair trial. Aside from the blast-shattered area which complicated the situation, the grouting work at Estacada was continually interfered with and hustled about, and hurried beyond the possibility of proper conduct, by the exigencies of rush construction; and, in the opinion of the writer, the engineers in charge of the grouting did remarkably good work under the circumstances. If, as the author states, no leaks of consequence have been observed below the Estacada Dam, may not a certain amount of credit be given to that part of the 1 920 bbl. of cement which was not wasted in handling surface leaks, and is it not conceivable that, had the work been less

rushed, and had greater care with the grouting been possible, the flow in the test holes inside the dam might have been less? Mr. Hulse.

Grout work is likely to be at a disadvantage when brought into conflict with the man whose sole idea of the conduct of work is to set a new record for depositing a given quantity of concrete in the shortest possible time. For the grouting, there is little in evidence but a few small pipes—which may or may not connect with an underground opening which might, later, imperil the integrity of the concrete man's work, when the reservoir fills—and these small, insignificant pipes may even get in the road of the concrete man's buckets, and otherwise become very much of a nuisance.

If, therefore, it is possible to do the grouting and get the pipes out of the way before the beginning of actual construction, so much the better. If the grouting and construction must be carried on simultaneously, every effort should be made to keep them from interfering with each other; but, in case they must conflict, it seems rather superfluous to point out that, inasmuch as the integrity of the whole job will depend to a very great extent on the success of the grout work, the latter should be given at least "an even break" in the matter of precedence. An open cut-off trench, by its very presence, compels respect and consideration, but a line of 2-in. grout pipes, which may be expected to accomplish cheaper and better results than the cut-off trench, must frequently be aggressively guarded by the man in charge of them.

In any case, it is desirable that provision be made for drilling and grouting after the completion of the work, should later developments show the need for this later work. Usually, this may be done more at the cost of foresight than at the cost of trouble and expense.

The author's suggestion about the drilling of a row of holes close together and then broaching them into an open slot which may then be grouted, is open to the objections against blasting in the foundation material under the heel of a dam to be. If, however, this might be accomplished without damage, the writer would suggest that, before the holes are broached, they should first be carefully grouted so as to silt up the adjoining territory, and thus reinforce the thin solid cut-off to be made by filling the slot. Further, it would be most desirable, in filling the slot, to do so by introducing the grout at or near the bottom of the slot, and forcing it to rise slowly toward the surface. A succession of such operations, to be accomplished in a deep slot, by a gradual raising of the grouting pipes, would probably insure the tightest filling of the slot with the least chance of subsequent shrinkage and cracking of the material deposited therein. Without some reinforcement, which might be very difficult to place in a thin wall like this, the writer would not consider the scheme except in connec-

Mr. Hulse. tion with a careful silting up of adjoining territory by first grouting the holes, as has been stated.

As a general proposition, the writer believes that the silting up of a foundation by grouting and by the action of the water impounded behind the dam, offers possibilities distinctly preferable to those of the conventional cut-off wall. In the case of a clear stream, like the Clackamas, it is usually practicable to make the water muddy by dumping or sluicing into it material from the banks, while the reservoir is filling, and thus close many or all of the openings missed by, or too fine for, the grout. The practice of shooting into foundation material under the heel of a dam is more than likely to make a lot of trouble—witness the left channel at Estacada—and the penetrating qualities of cement grout, properly handled, are very great. If the rock is good, why replace it with a concrete cut-off? If the rock is bad, and yet so solid as to necessitate blasting for its removal, how much better to silt it tight and leave it undisturbed?

Mr. Fisher. FRANK R. FISHER,* ESQ. (by letter).—In the writer's final report, as Resident Engineer in charge of the construction of the Estacada Dam, he stated, in discussing the grouting of foundations, that an impervious cut-off was not accomplished, that is, to the absolute degree had in view when the method was adopted. The varying and erratic nature of the formation made the solution of the problem largely experimental and one that could only be worked out by a thorough trial and without any positive assurance as to what the final outcome would be.

The writer does not consider that the method should be condemned or judged by the failure to obtain absolute tightness in the very peculiar material with which the engineers had to deal, but rather that credit should be given them for reducing the possibility of seepage to the extent that practically no uncertainty regarding the safety of the foundations in that respect would remain. That this was accomplished, and the foundation material benefited by the treatment, he believes an analysis of the results, as stated in his report, will show.

Mr. Rands arrives at the following conclusion: "that over a portion of the cut-off no grouting was needed, and that over other parts where it was needed it did little good." The first part of his statement is correct, in that over a considerable portion of the right channel and island sections, the average formation was found to be in good condition, not absolutely tight, but developing only a low rate of seepage under test. Naturally, this evidence was not apparent until it was disclosed eventually by the progress of the work, and then, in view of the ever-present uncertainty, due to the fact that occasional points would develop unexpected weakness, it was deemed advisable to fol-

* Lansdowne, Pa.

low out the original plan to completion. In the light of final knowledge acquired from the complete data obtained, it was evident that the grouting could have been omitted over certain stretches, and the number of holes on these sections materially reduced. **Mr. Fisher.**

The remaining section, that across the left channel, is evidently what Mr. Rands refers to as: "other parts where it was needed it did little good". There was no question as to the need of some kind of treatment in this particular locality, as the material, from the standpoint of permeability, was the worst met with on the site of the dam. The water-pressure tests gave high rates of seepage, communication existed between holes located over a considerable distance, and also with the surface as many as 70 ft. from the point of testing. The holes took grout freely and, in the aggregate, several hundred barrels of cement were forced into the seams and crevices existing throughout the rock. As the individual seams were apparently of little width, the large quantity of grout taken would seem to indicate that it was diffused over a wide area.

Although the tests on the final proving holes, at the full depth of 50 ft., developed considerable seepage, it was very much less than the original. The most encouraging feature, however, was the very satisfactory results obtained for the first 30 ft. Over a length of cut-off of 70 ft., the aggregate of the seepage through thirteen holes, spaced approximately 6 ft. apart, when tested under a hydrostatic head slightly in excess of that created by the dam, was only 0.2 sec-ft., and it should be noted that this seepage was obtained by boring into the vitals of the foundations, and opening up and subjecting to direct pressure all points of weakness intercepted by the thirteen holes. It should also be kept in mind that this test was made on what was originally the weakest section of the entire cut-off.

It is also obvious that the test conditions were much more severe than could occur normally from the pond pressure, and, therefore, it is reasonable to conclude that the grouted cut-off is effective for a depth of 30 ft., and as this is reckoned from the bottom of an overlying 10-ft. concrete wall, the total effective cut-off is 40 ft.

As to the choice between a concrete and grouted cut-off for this depth, it is only necessary to point out that the high cost, difficulties of construction, and the element of time required—of vital importance in this case—would practically prohibit the use of concrete.

Evidence of the satisfactory final condition of the foundations was furnished subsequent to the completion of the dam and the filling of the pond by a thorough examination of the surface of the rock inside the dam, and the unwatered bed of the river immediately down stream. No springs or seepage could be observed, with the exception of a few very slight indications along the side of the left bank, and it is more than likely that these were from surface drainage.

Mr. Fisher. The foregoing facts may not offer indisputable proof that the final satisfactory condition of the foundations was entirely due to the merits of the grouting treatment, but the evidence is in its favor, and it is entitled to all the credit that the results seem to warrant.

Mr. Coburn. H. L. COBURN,* M. AM. SOC. C. E. (by letter).—Mr. Rands is to be congratulated on the thorough manner in which he has presented the problem and attempted solution in the matter of the cut-off under the Estacada Dam. The information contained in this paper will be of great interest to the Profession. There is little which can be added to Mr. Rands' statements, and the writer thoroughly agrees with his conclusions as to this particular structure.

In such a foundation as was found in Estacada it seems that the only reason for making extraordinary efforts to secure an impervious cut-off at great depth would be for the purpose of saving water, and that it was not at all necessary to prevent undermining, as the rate of flow through this foundation was so slow as to be negligible. If, however, some further protection was deemed necessary, the writer believes that the concrete cut-off could have been carried very considerably deeper than was done for a lesser sum than was expended on the grouting, and with much more definite assurance of success. If, in addition to this concrete cut-off, a carpet of fine silt, either of clay or volcanic ash or other fine material—which is present in large quantities—had been sluiced into the river bed and against the dam and extending several hundred feet up stream, it would appear certain that a very satisfactory seal would have been obtained. Indeed, the writer is confident that the natural deposit of silt which is inevitable in such a pond will seal this structure very completely, and in a comparatively short time.

Mr. Cushman. WILLIAM H. CUSHMAN,† M. AM. SOC. C. E. (by letter).—The general scheme had been adopted and some of the earlier drilling accomplished at the time the writer took general charge of the project. The execution of this portion of the work, as already determined, became a duty, regardless of an individual lack of confidence in the ultimate successful accomplishment of the end in view.

That the foundation material was sufficiently good to carry all the pressure to which it could possibly be subjected, will be conceded by all engineers familiar with the designs for the dam and the foundation material.

Although the breccia was porous, it was substantially free from open cracks, and it was scarcely to be considered a case where the percolation of water through the foundation material would result in undermining the dam.

* New York City.

† Scotia, N. Y.

Admitting these two propositions to be true, it would seem that the only practical consideration would be the one mentioned on pages 479 and 480, that is, whether the water lost by percolation would be valuable enough to justify the cost of the grouting work. Mr. Cushman.

Although the Clackamas is a clear-water stream, undoubtedly some silt depositing could be depended on, and possibly selected materials could have been deposited in the pool above the dam to assist to a large extent in silting up the porous rock, etc.

Throughout the paper (as mentioned on page 464, the seepage test detailed on page 466, and again on page 472, where serious thought of discontinuing grouting was entertained), considerable skepticism is expressed as to the effectiveness of the method of treatment. The quoted opinion of the author, that "Over those parts where it was needed it did little good", will be commonly agreed with, and cause little surprise to those who have had similar problems to solve.

The suggestion mentioned on page 481, that the rock between the rows of drill holes be shattered with small charges of powder, originated with the writer, and it still seems that such shattering might have facilitated the flow of "thickened grout" and made it possible to restrict the treated area to the immediate vicinity of the cut-off wall, rather than diffuse it to remote points by subjecting it to the heavier pressure required to force it through small openings.

It would be interesting to know whether an attempt has ever been made to shatter a trench through rock, to a depth of as much as 50 ft., and then introduce grout or thin mortar (fed through pipes from the bottom and flowing upward or otherwise) in such manner as to produce, in effect, a concrete wall to the depth stated. It would not seem impracticable to accomplish this, and do away with surface capping with concrete and the use of excessive pressure.

Considerable thought and investigation were given by Messrs. Rippey, Fitzgerald, Schreiber, and the writer, to various methods of cutting an open trench or seam through the rock to the required depth, in order that an impervious curtain could be introduced. Special channeling machines, drill holes closely spaced and then broaching out the material between holes, and even open cut, were considered. It would be of interest to hear from engineers who have accomplished something along these or similar lines.

Mr. Rands is to be commended for the painstaking manner in which he has presented this interesting subject, and, though it be admitted that the procedure was, to a large extent, futile, nevertheless, his subsequent work has proven the value of this first experiment.

V. H. Hewes,* M. Am. Soc. C. E.—The speaker would be glad if some member of the Board of Water Supply, who is familiar with the grouting work used in the siphons on the line of the Catskill Water Mr. Hewes.

* New York City.

Mr. Hewes. Supply for New York City to check the excessive inflow of water during construction, would describe the method used in that work.

The speaker was granted the privilege of visiting the inverted siphon under the Hudson River where a heavy inflow was encountered, which threatened to drown out the pumps, although extra pumping facilities had been provided to care for such an emergency. A bulkhead was placed in the drift near the heading; pipes were introduced through the bulkhead, and grout was pumped through them, thus effectually reducing the inflow. After the bulkhead was removed and the work of drifting was continued, it was found that the cement had filled the rock seams both laterally and longitudinally. A sample of porous rock was shown which had been taken from another siphon on the line, where the grouting method was used to check the inflow. This sample seemed to be thoroughly impregnated with cement.

A description of the method used to relieve the water pressure on the outside of the concrete lining until the concrete had thoroughly set at points where there were small seams in the rock walls, and the subsequent grouting of these seams, would be exceedingly interesting.

Mr. White. LAZARUS WHITE,* ASSOC. M. AM. SOC. C. E.—It is interesting to compare the results at the Estacada Dam with those obtained on the Catskill Aqueduct. The Ashokan and Kensico Dams have very deep concrete cut-offs. Some grouting was done below these cut-offs, but only as a secondary matter. In the sinking of the shafts there was considerable grouting, which afforded a good opportunity to see what it would do, because, in sinking a shaft, the portion grouted is subsequently excavated. The grouting of tunnels during driving also gives very definite information. The grouting work on the Catskill Aqueduct has been going on for 5 or 6 years, and has been watched for this entire period, so that observing engineers have come to some conclusions as to its results. These conclusions are remarkably similar to those expressed in this paper, which is exceptionally instructive.

The lack of success of a portion of the Estacada grouting is frankly admitted. It seems that in the last few years grouting has been made a sort of fetish; that is, people have been expecting too much from it. At best it is an uncertain proposition, and should be secondary to the customary work. The same applies to the excavation of tunnels. More water is cut off by the concrete lining than by the grouting. Grouting at times is wonderfully successful, but the conditions then are rather favorable; where the rock is hard, with definite splits and channels, the grout can be forced into it readily, but where it resembles the volcanic rock described by Mr. Rands the grout cannot be forced in thoroughly, but takes its own channels. It may run a long way from the part to be grouted, and on excavating this part, very little of it will be found. No one knows where it went—all that

* New York City.

is known is that one has paid for it. At some shafts on the Catskill Aqueduct holes have been repeatedly grouted, and when the excavation was extended through the grouted zone the grout could hardly be seen—the same being even more true of grouted tunnels. Mr. White.

As a general proposition, the grout goes into places which the engineer has good reason to know exist, that is, into definite cracks or openings. At some shafts grouting has been very successful, but, as already mentioned, the conditions there were favorable. Where unfavorable conditions were found, for instance, resembling those at the Estacada, grouting was not very effective. Judging by experience on the Catskill Aqueduct, the conclusions of this paper are entirely sound.

Grout always seems to obtain some set, but if it is forced into a place where there is water under pressure, and if the water is circulating, it will carry the grout out of the seams before it has a chance to set. Grout has been chased over tunnel arches, up and down, for hundreds of feet. Before the grout can set, it may be forced out, and pipes and seams which were thoroughly grouted may become empty, but if the grout is kept in place long enough, it will set, no matter how much water is put in. This seems to be the speaker's experience.

FREDERIC V. ABBOT,* M. AM. SOC. C. E. (by letter).—Mr. Rippey's discussion is exceedingly interesting, because it carries to much further length a method adopted by the writer in 1899, in constructing a cut-off wall of moderate depth at Lock and Dam No. 2, Mississippi River, between St. Paul and Minneapolis. At this locality the material forming the bed of the river was a very hard combination of clay, sand, and limestone fragments, mixed with a few granite boulders, overlying the well-known, friable sand rock of this part of the Mississippi Valley. Mr. Abbot.

Fig. 19, shows the nature of this material, which underlaid the lock floor and the lock walls. A description of the difficulties encountered in handling this material, which is quickly eroded by rapidly flowing water, and of the methods adopted to overcome them, is given in the following extract quoted from the writer's Annual Report to the Chief of Engineers, U. S. Army, dated July 17th, 1900:

"The seams were all originally water-bearing, and only became dry after long pumping had drained out all the water tributary to them. At a lower level these seams run out into the bed of the river, and in this way passages for water are opened between the river bed and the interior of the cofferdam, many feet below the original bottom of the river at the point where the cofferdam stands. It had been realized from the start that such seams were to be encountered, and to avoid opening up too great a flow at any one time a novel form of cut-off wall was adopted after prolonged study. A slot in the sand rock is made, 2 inches wide and 12 feet long, parallel to the river wall and lying between two longitudinal foundation timbers, by jetting

* New York City.

Mr. Abbot. a series of intersecting vertical holes 10 feet deep and breaking off any projecting edges between the holes. Most of the eroded material is brought up out of the slot by the jet, and the rest is easily pumped out with the ordinary Edson diaphragm pump and two men. In the usual case this leaves a perfectly clean slot with a considerable flow of water pouring out of it, the water coming from the underground seams which connect with the river. Into this slot a diaphragm made of tongued and grooved three-quarter-inch boards, in two layers breaking joints, is lowered by hand. Three vertical 1½-inch iron pipes, extending down into the slot only a few inches, are inserted, one at each end and one at the middle of the slotted section. A steam pump connected to one of these pipes is now started just fast enough to take care of the flow from the slot without letting it run over, or draining it below the level of the surface more than a few inches, and concrete is rammed into the space above the slot and between the foundation timbers above referred to, which have been firmly drift-bolted down to the sand rock before the slot was cut. As soon as the concrete has set—that is, in about twenty-four hours—the pump is stopped, and water at once flows out of all three pipes. Additional lengths of pipe are screwed on the three till the flow is stopped—that is, till the tops of the pipes are at a higher level than the head due to the pressure in the seams. Thick grout is now poured into smaller pipes passing through the first pipes and reaching to the bottom of the slot. As the latter fills with grout the water is forced back into the seams. The small pipes choke after the grout rises a couple of feet above their lower ends, and they are then raised till they again flow freely. When the slot is full they are removed and the larger pipes are kept filled to the top with the grout as long as any more can be poured in. After about twenty-four hours all the grout has hardened, the pipes are removed, cleaned out, and used over again in another section of cut-off. (Photographs Nos. 7, 8, and 9* show an experimental section of such a cut-off wall which was put in at a high level in the lock pit and then carefully excavated.) The perfect grout filling of the most minute

* Reproduced herein as Figs. 21, 20, and 22, respectively.

Note on Fig. 19.—The extreme friability of the material is shown. The method of supporting the timbers on this unreliable material is shown at the top of the picture. The timber is wedged up on wooden blocking so as to leave a space of about 2 in. between it and the surface of the sand rock. The anchor-bolts are then driven and the wedges removed, leaving the timber supported on the anchor-bolts. The surface of the sand rock is then thoroughly washed with water from a hose, all loose matter is removed, and the space between the fresh sand rock surface and the timber is filled with good concrete rammed in place with heavy sledges and steel followers.

Note on Fig. 20.—The two large timbers between which the concrete is seen represent foundation sills. They are fastened to the sand rock every 4 ft. by the 1¼-in. drift-bolts, of which one is seen extending down into the wooden pin, a part of which has been removed. These bolts extend 3½ ft. into the wooden pins, and require a force of more than 9 tons to withdraw them. The wooden pin to the left is shown encased with grout, as it was when the sand rock was carefully removed from around it. The narrow white column about half way between the two pins is the end of the cut-off wall as it appeared when the first 2 ft. of the sand rock was removed. The white marks, looking like splashes of whitewash, are really thin filaments of grout which had penetrated the seams in the sand rock, showing that the cut-off wall is not only a thin impervious vertical curtain in the sand rock, but that the latter itself is solidified for at least 2 ft. on each side. The board in a horizontal position under the concrete is used to prevent the fresh concrete from falling down in the slot alongside of the board curtain, and thus preventing the grout from having a thorough subsequent circulation.

Fig. 19.—Vertical Face of Sand Rock on the Lower Side of the Excavation for the Upper Lock Gate and Culverts.

Fig. 20.—Experimental Section of Cut-off Wall, Partly Excavated, to Determine Whether the Grout Filled all Crevices, Etc.

FIG. [illegible].—[illegible]

FIG. [illegible].—EXPERIMENTAL SECTION OF CUT-OFF WALL PARTLY EXCAVATED TO DETERMINE WHETHER THE GROUT FILLED ALL CREVICES, ETC.

cracks in the sand rock is surprising. An absolutely tight cut-off wall from the bank above the lock to the outer toe of the river wall, and under this toe for its full length and back to the bank below the lock, extending to a depth of 23 feet below dead low water in the river, or deeper in that part under the river wall, is thus secured. Mr. Abbot.

"In some cases no subterranean seams are encountered, and then the wooden diaphragm is inserted and the grout at once poured in around it through pipes leading to the bottom. Experience has shown that in such cases time is a vital element. A slot which does not develop the least flow at first, has in several cases developed very great and troublesome flow in twenty-four hours if not grouted. Most trouble was encountered at the upper end where a very large seam directly connecting with the river had to be cut off. This seam extended nearly horizontally under a space fully 30 feet wide by 80 feet long, and every section of cut-off wall intersected it. Grout poured into one section would pour out of another 30 feet distant. As these underground connections were not suspected, the quantity of water which poured out of three openings into this seam soon exceeded the capacity of the pumps. All arrangements for grouting two of the three sections were completed except the stopping of the flow from the pipes, which in this case were 6-inch in diameter and were running full. It is believed to be unsafe to try to stop suddenly such a flow for fear of blowing up the lock floor. The cofferdam was allowed to fill, thus gradually stopping the flow from the seam, 6-inch risers were screwed into the now submerged overflow pipes and both sections were grouted at the same time from staging erected above the water level. Twenty-one barrels of dry sand cement were required for the grout to fill the slots and seam. In three days the cofferdam was again pumped out and the two springs were found stopped entirely. The third was really outside of the lock walls, being in a section of slot that had been put in to make connection with the cut-off wall under the dam. It was successfully separated from the cut-off slot under the lock wall, which has since been easily grouted."

In the preceding paragraph mention is made of sand-cement. In view of the high freight rates existing at that time between the American Portland cement factories and St. Paul, sand-cement was used in the construction of the land wall of this lock. To insure getting exactly the proportions of sand in the sand-cement, a tube mill was purchased and the cement (Saylor's brand) was ground at the lock site with a very excellent silicious sand found in the vicinity. The concrete made with this sand-cement has stood the extreme temperature conditions at St. Paul, 105° in summer to 40° below zero, Fahr., in winter, for the past 14 years in a surprisingly satisfactory manner. The river wall was built of Saylor's cement, used in the same proportions as the sand-cement in the land wall. A very sharp comparison of the two kinds of concrete has thus been secured. Although the writer has not been able to inspect the work, he has been told, by those who have, that the concrete in the land wall is, if anything, superior at the present time to that in the river wall. For

Mr. Abbot. the land wall, the cement was ground with an equal proportion of sand, by volume, and the sand-cement thus produced was used in the same ratio to unground sand and broken stone as if it had been regular Saylor cement.

In connection with the construction of this lock, a great deal of grouting of seams similar to that described was necessary, and in this work great economy resulted from the use of sand-cement in place of undiluted Saylor cement. Experiments were made in order to determine whether sand-cement applied in the shape of grout would harden. A vertical glass tube, 4 or 5 ft. long and about 1 in. in diameter, was plugged at the bottom and filled with a 1 to 1 sand-cement grout and allowed to harden. The tube was broken in 3 days, and it was found that the lower part of the grouting in this pipe was as hard and sound as that at the top. When ordinary sand was mixed in the same proportion with undiluted Saylor's cement, kept thoroughly stirred up, and poured into a similar tube, the sand settled to the bottom very rapidly, leaving a creamy grout with hardly any sand grains in the upper part of the tube. Under these conditions the bottom 2 ft. never set, though the upper 2 or 3 ft. set about as hard as the sand-cement grout did in the other tube. The reason is plain. The specific gravity of cement clinker is not widely different from that of silica. When the silica is in large grains, and the cement clinker is ground fine to make a Portland cement, the two materials separate in water, the larger particles settle quickly and reach the bottom first, and the finer particles, which settle less rapidly, remain near the top of the column. Since making this experiment the writer has always had considerable doubt as to results of grouting with Portland cement stirred up with ordinary sand, but he has no doubt of the effectiveness of grouting with sand-cement ground 1 part of cement to 1 part of silicious sand, provided the grinding is carried far enough to make the sand as fine as ordinary Portland cement. From Mr. Rippey's description, the writer supposes that no sand was incorporated in his grout. If sand-cement had been used, it would seem, therefore, that the cost of grouting might have been somewhat reduced provided the freight rates to the Estacada Dam were high enough to make it pay to install a cement-grinding tube mill.

Note on Fig. 21.—The complete covering of the end of the wooden curtain by the grout is well shown, as well as the way in which the latter filled every irregularity in the end of the original slot. The grout was about 1 week old, and was at the time firm and hard. It was composed of sand-cement ground in the proportion of 1 sand to 1 Saylor's Portland cement. The extreme wetness of the sand rock is well seen in the pond of water at the foot of the section. Though there was no regular flow, the water was oozing from all the pores of the face exposed by the excavation.

Note on Fig. 22.—At the top, the grout covering the end of the diaphragm has been removed to show the boards, and, in the middle of the plane parallel to the diaphragm, the grout has been scraped off to show the flat side of the boards. The rest of this plane is left as it stood when excavated, showing that the grout absolutely holds the sand rock to the boards. The irregular projections at some points in this plane show where there was originally a seam in the sand rock, which has been filled by the grout.

EXPERIMENTAL SECTIONS OF CUT-OFF WALL.

FIG. 21.—WHEN EXCAVATION HAD PROGRESSED TO A DEPTH OF 6 FEET.

FIG. 22.—A SECTION PARALLEL TO THE DIAPHRAGM OF BOARDS, AND A FEW INCHES, ONLY, FROM IT.

J. F. RAMSBOTHAM,* M. AM. SOC. C. E. (by letter).—The writer has read this paper with great interest, and is of the opinion that it will be of value to the entire Profession. He would like to draw some conclusions based on his experience gained on the Fremantle Graving Dock.† Mr. Ramsbotham.

In the first instance he differs with the author in regard to the method of starting with a low pressure of 25 lb. and ending with a pressure of 250 lb. per sq. in. Surely the pressure should have been higher in the initial stages, for, with the depth of hole in use, there was no fear of doing damage, and, at the same time, a better and more pronounced penetration would have been assured.

A pressure of 250 lb. per sq. in., or 16 tons per sq. ft., is certainly very high, and the quality of the rock must have been exceedingly tough to have withstood it. The danger of using such a pressure is considerable, but risk of damage would be lessened by having other holes drilled in the vicinity of the grouting, such holes acting as a safety valve.

The author's summary of the advantages reaped is somewhat disappointing.

From the writer's experience, cement grouting can be relied on to form a cut-off from percolation water, and this experience was gained by witnessing daily a material decrease in pumping.

The holes should be staggered, the front row being 6 ft. from center to center and the back row 3 ft. back from them. The writer considers it preferable to drill the front row first and then check and test the results by the back row, but it is quite impossible to lay down any hard and fast rule. Each case must be judged on its merits.

The whole question of efficiency is problematical, and for that reason, before adopting cement grouting and giving a positive opinion, an actual inspection of the site is desirable.

As regards fortifying bad ground (such as a gravel bed) to withstand a heavy statical load, the writer has no hestitation in saying that by cement grouting this can be done economically and safely, there is no excavation to make and replace with concrete, and no pumping.

HARRISON SOUDER,‡ M. AM. SOC. C. E. (by letter).—It is a fact that very little has been published in regard to the treatment of foundations by the use of cement grout. The interest manifested in the Estacada work causes the writer to regret deeply that his experiences in this line some 13 years ago, were not given to the Profession at the time. However, the identical method of securing a tight cut-off wall—by injecting a cement grout under pressure through drill holes, as de- Mr. Souder.

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* Melbourne, Victoria, Australia.

† *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1942.

‡ Cornwall, Pa.

Mr. Souder.

scribed by Mr. Rands—was used by the writer in the construction of the Hinckston Run Dam at Johnstown, Pa., as long ago as 1901.

The fact that this method of grouting was used in the foundations of this work prior to the Estacada operation is brought out in a brief description of the Hinckston Run Dam published in 1909 by the late James D. Schuyler, M. Am. Soc. C. E.*

At the time this method was adopted at Johnstown, the only information the writer had on the subject was that gained from watching the grouting of the tunnel backing of the deep sewers built in connection with the Pennsylvania Avenue Subway and Tunnel and in closing leaks in the Torresdale Conduits, in Philadelphia; and especially from a report† by F. V. Abbot, M. Am. Soc. C. E., Major (now Col.) Corps of Engineers, U. S. A., Engineer in Charge of Improvements of the Mississippi River at St. Paul and Minneapolis. The report describes an extremely interesting method of forming a tight cut-off wall by using cement grout, exactly after the method suggested by Mr. Rippey on page 495 and illustrated by Fig. 16, except that the diaphragm was made of wood and the grout was applied under hydrostatic pressure sufficient to overcome the head of the water flowing through the rock seams.‡

As the work at Johnstown was in the seamy shales of the soft-coal formation, a detailed description of the cut-off wall may be of value even at this late day. To add to the clearness, and as a matter of record, a brief description of the dam is given.

The Hinckston Run Dam.

In 1900 the Manufacturers Water Company, after a comprehensive study of the problem of increasing the water supply of Johnstown, Pa., and the rapidly expanding plant of the Cambria Steel Company, determined on the construction of several large reservoirs and pipe lines.

In view of the frightful calamity of 1888, caused by the breaking of the South Fork Dam, there was naturally much opposition to the construction of any new dams above the city. Under the direction of Mr. John Birkinbine, as Consulting Engineer, the first of the proposed reservoirs was located 5 miles from Johnstown, on Hinckston Run, a stream which flows into the Conemaugh a short distance below the famous Stone Railroad Bridge. All the streams in this district are essentially mountain streams. The valleys are narrow and the hillsides steep and rocky, and therefore the run-off is rapid. The average yearly rainfall is 36 in., but it is not well distributed, the major part coming down in the winter and spring; though some extremely heavy and sudden downpours occur in the summer.

* "Reservoirs for Irrigation, Water Power and Domestic Water Supply," p. 518.

† An abstract of this report was published in *Cement*, January, 1901.

‡ The reader is referred to Col. Abbot's discussion, which contains an extract from his report, and several illustrations.

The original Hinckston Run project called for an earth dam, 60 ft. high, to retain some 400 000 000 gal. of water, with a depth of 45 ft. at the breast. **Mr. Souder.**

The intention, as stated, was to build a dam 60 ft. high, with a clay core, but, as an unlimited quantity of cinder from the steel plant was available, it was decided, after the work was started, to use this as backing for the dam, in place of earth, and eventually to fill the whole valley below with this material, thus rendering the structure practically unbreakable. In view of this and the additional expense incurred in making the cut-off tight, the proposed height of the dam was increased to 80 ft., and later to 85 ft., above the original creek level. This gave a total maximum height above the bottom of the core-wall ditch of 112.8 ft., a depth of water at the breast of 73½ ft., and a capacity of 1 100 000 000 gal. The lake thus formed is 1¼ miles long. The water-shed above the dam is 10.75 sq. miles.

With the consequent great increase in pressure, extra care was needed, and every precaution was taken to make the structure water-tight.

The site selected for the dam required a rather long breast, but gave an excellent opportunity to construct a spillway with natural rock bottom, and was close to several large deposits of excellent clay lying within the reservoir site.

Work on this dam was started in the fall of 1900. The writer took charge as Resident Engineer and Superintendent of Construction early in May, 1901. At that time the dam site had been cleared of timber and partly stripped, and a small section of core-wall ditch had been excavated to a depth of 12 ft.

The first work done was to strip the surface of the dam site of all soil and vegetable matter. This was generally 3 ft. deep, but over a large area, near the creek channel, it was necessary to remove material to a depth of 8 or 9 ft. owing to numerous beds of muck, old logs, etc. All this material was deposited in the rear of the embankment proper. The remainder of the reservoir area, some 105 acres, was cleared of all timber and brush, but the surface soil was not removed, as the water to be stored was intended for mill purposes only.

Work on the dam proper was suspended on rainy days and during the winter.

The cross-section of the dam as built is shown by Fig. 23. The lower inner slope is 1 on 2½, with 4 ft. of puddle and 24 in. of cinder rip-rap. The slope above the berm is 1 on 1¾ with puddle lining diminishing to 2 ft. thick at the top. The facing is hand-laid stone paving. The puddle wall is 16 ft. thick at the top of the concrete core-wall, and diminishes to 4 ft. at the top of the dam.

Early in July, 1901, when the core-wall ditch had been carried down to hard rock at what was thought to be the proper depth, some test

Mr. Souder. holes were bored through the bottom to determine the character of the rock below. This disclosed a layer of hard sandstone a few feet down, with considerable water flowing below and above it. It was decided to deepen the ditch considerably, in order to get below any rock strata that might come to the surface within the flooded area, and to substitute a concrete core-wall for the clay one originally proposed.

An air compressor plant was installed. This was a 14 by 18-in. Ingersoll-Sergeant machine capable of driving two rock drills and six pneumatic rammers. These rammers were used in tamping concrete and also in puddling clay in such places as could not be covered by a 10-ton steam roller which was also supplied at this time in place of the 3-ton horse roller in use.

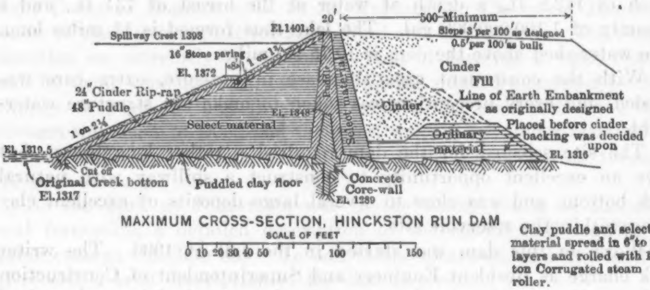


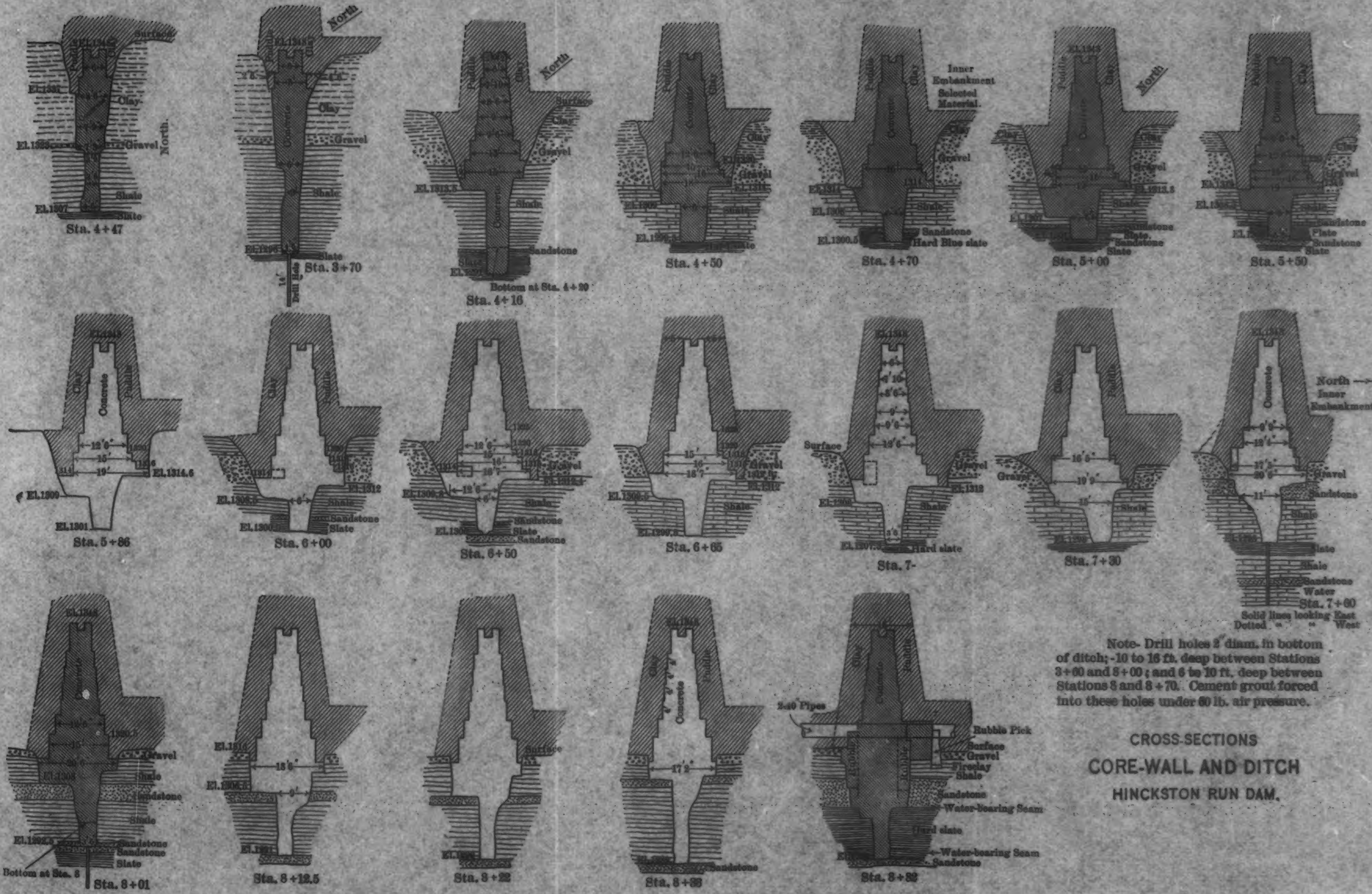
FIG. 23.

The finished ditch averaged in depth 25 ft. below the grubbed surface in the valley, but reached 50 ft. at the ends. The shale was excavated with picks, but the harder rock was loosened with light charges of dynamite, care being taken not to shatter the foundation or open up the seams. Plate XI shows typical sections through the core-wall and ditch, the variable character of the strata cut through being indicated. The advisability of cutting off the underflow to as great a depth as possible was realized, and it was determined to remove the shale down to the sandstone and try to cut off the flow below by forcing in cement grout under air pressure.

Holes, 2 in. in diameter and from 10 to 16 ft. deep, were drilled through the rock, averaging about one hole per linear foot across the valley. Iron pipes, 2 in. in diameter, 18 in. long, and threaded on one end, were cemented into these holes. Portland cement grout was poured into them and then air at a pressure of from 30 to 60 lb. was applied.

The first holes were approximately 6 ft. apart. They were drilled generally 10 ft. below bed-rock. The first hole drilled was marked No. 1 W., the second No. 2 W., 5 ft. distant, the third No. 4 W., 17 ft. distant.

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1318.
SOUDER ON
GROUTED CUT-OFF.



Note- Drill holes 2" diam. in bottom of ditch; -10 to 16 ft. deep between Stations 3+60 and 8+00; and 6 to 10 ft. deep between Stations 8 and 8+70. Cement grout forced into these holes under 60 lb. air pressure.

CROSS-SECTIONS
CORE-WALL AND DITCH
HINCKSTON RUN DAM.

After No. 1 was grouted, a small pit, or sump, was excavated through bed-rock and into the shale below. Signs of grouting were visible 2 ft. below the sandstone, which was encouraging. After some twelve holes had been drilled within a distance of 66 ft., an effort was made to determine the location and extent of the water seams below bed-rock by using a solution of permanganate of potash. In a letter to Mr. Birkinbine, dated July 22d, 1901, the writer described this test as follows: Mr. Souder.

"In the main core-wall ditch I made some tests to determine the location and length of seams under bed-rock. Mr. Hyde, the Cambria Chemist, brought some potassium permanganate out. We put several buckets full of the solution into the receiver and connected it with hole No. 3 West and then turned on the air. The color showed in the sump and in holes Nos. 4 and 5 and slightly in No. 6, entering the last hole at least 2 in. from the surface. This showed that 2 in. and 3 in. of shale had not been stripped from bed-rock, also that the seam extended from No. 3 hole beneath bed-rock to the sump. This test was not altogether satisfactory, so we put a pipe in No. 6 W. hole, from which a strong clear stream of water flows. Put some permanganate in the tube and connected the air hose directly to the tube. The color showed in Nos. 4 and 5, in fact, as far as the sump. About this time the pump valve got out of order and the water rose in the trench, covering most of the holes. Then I put on full air pressure, and after a time it began to appear all along the trench bottom from the sump to hole No. 12. Up to No. 10 it was very distinct, at Nos. 11 and 12 the effect was not quite positive. Keeping the pressure on, I had the sump drained, and found the air blowing in at the bottom and on the far side and lower side.

"Evidently there are seams underlying the length of bed-rock. The small *y*'s on the following diagram [Fig. 24] indicate where the air blew out."

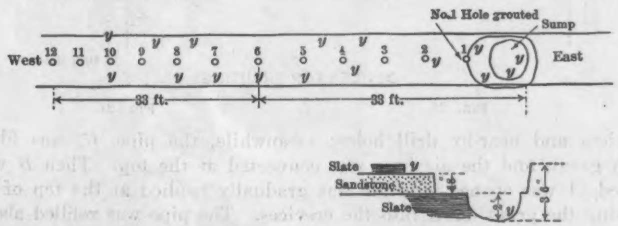


FIG. 24.

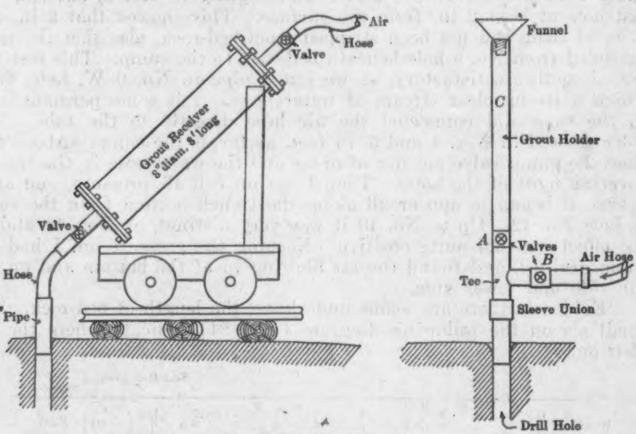
Fig. 25 is a sketch of the first contrivance or receiver devised for applying the grout. It was a cylinder, 8 in. in diameter and 6 ft. long. A screw flange was provided at top and bottom, and a steel head-plate was bolted to each end, with rubber gasket packing. The top bolt holes were open to allow quick removal of the lid. A 2-in. pipe with plug cocks was provided at the top and bottom. With a short hose, the cylinder was coupled to the pipes in the holes. The

Mr. Souder.

cylinder was filled with grout; the valve was opened; the grout ran into the drill holes, and air pressure was then applied at the top. The contrivance was mounted on a truck running on a track in the bottom of the ditch. After trial it proved to be too slow and cumbersome, and another method was devised and operated satisfactorily. Fig. 26 is a sketch of this final arrangement, and Fig. 27 shows it in operation.

The method of grouting was as follows:

A 1-in. pipe, long enough to reach to the bottom of the hole, was inserted and air was applied to blow out water and dirt. Then a tee and the pipe, *C*, were attached to the tube in the drill hole with a sleeve union, as shown. The cock, *A*, was closed, the cock, *B*, was opened, and air was applied, thus forcing the water out of the hole and into the



DEVICES FOR GROUTING

FIG. 25. FIG. 26.

crevices and near-by drill holes; meanwhile, the pipe, *C*, was filled with grout, and the air hose was connected at the top. Then *B* was closed, *A* was opened, and air was gradually applied at the top of *C*, forcing the grout down into the crevices. The pipe was refilled about every 10 min. until the hole would take no more. The apparatus was then removed and a cap was screwed on the tube in the grouted hole. After a given length of ditch was grouted in this way, and sufficient time had elapsed to allow the grout to set, test holes were bored within the grouted area and the process was continued until there was no indication of water flow below the bottom. The greater part of the bottom was grouted successfully in this way, but, as explained later, the grouting scheme was abandoned where the core-wall ditch entered the side-hills.

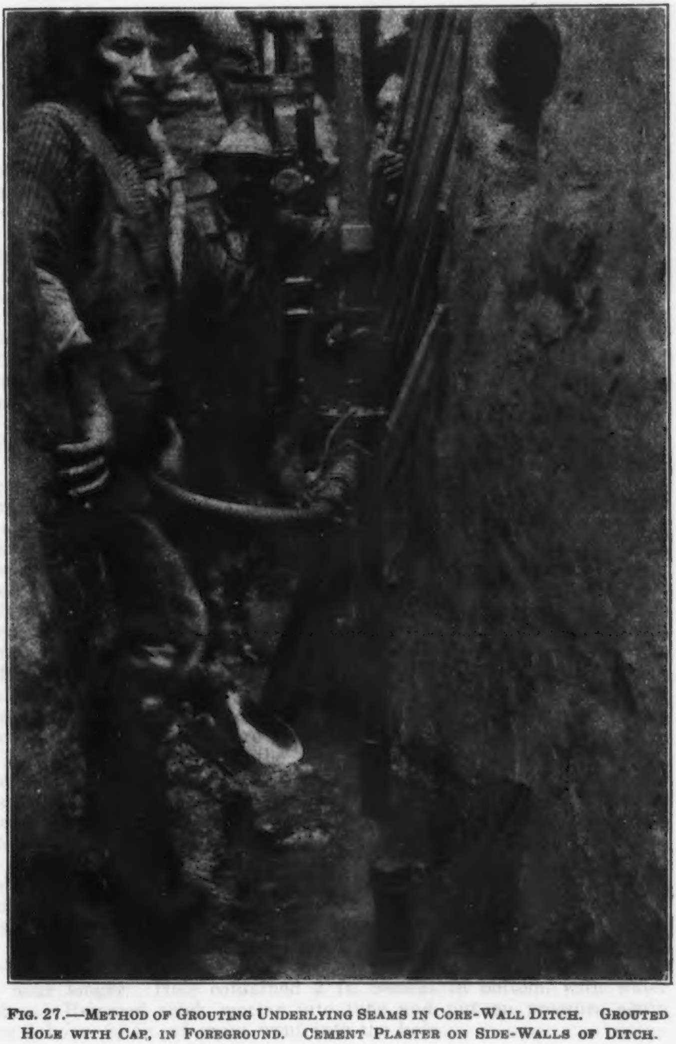


FIG. 27.—METHOD OF GROUTING UNDERLYING SEAMS IN CORE-WALL DITCH. GROUTED HOLE WITH CAP, IN FOREGROUND. CEMENT PLASTER ON SIDE-WALLS OF DITCH.

Descriptive of this work, the following extracts from reports by the writer, made at the time, may be of interest: Mr. Souder.

EXTRACTS FROM REPORTS ON GROUTING TESTS.

July 12th, 1901.—"Finished drilling second drill hole. Strip rock near first hole. No signs grout at 3 ft. distance."

July 13th, 1901.—"Excavated to hard rock and then drilled holes every 6 ft., 10 ft. deep. (Later, 16 ft. deep.)

"Cut away 18-in. layer of rock to within 2 ft. of first drill hole grouted and discovered evidences of grouting. Two of the new drill holes flowed water, which on test comes mainly from within 3 ft. of surface of rock."

July 17th, 1901.—"Test of setting of grout; after 2 hours the neat cement grout hardly set enough to resist anything like a current of water. Addition of sand helps it. Recommend quicker-setting cement."

August 3d, 1901.—"There is a thin layer of shale and slate on the sandstone at Hole No. 11 E. becoming thicker to the east at Hole No. 20; the sandstone is 6 ft. below the shale. This explains why water is found at two levels. On No. 19 E. we had water at 4 ft. and again at 15 ft. To-day grouted several holes. The first was No. 13 W. From this hole a fairly strong stream was flowing out of tube 12-in. high. I first forced air in the hole without cement. Air bubbled out of Hole No. 11 W. very weakly but quite strongly from Nos. 12 W., 14 W., 15 W., and 19 W. Made a smooth-flowing grout of 2 bags cement, 1 bag very fine screened sand, and 3½ buckets water. After the air had been on 30 min. a strong cement color appeared in Hole No. 15 and weaker color in Hole No. 14. Kept pressure on for 1 hour. The grout appeared rather soft in the hole, and after a time a very small thread of water came through it. Flow of water from No. 12 did not diminish appreciably, but carried slight cement color. Difference in level of grout in cylinder, before and after, 5½ in. Next grouted No. 11 W., forcing in 3¼ in. of grout after ¾ hour pressure, there was no perceptible difference in flow from adjoining holes. There was no water flowing from No. 11 before grouting. Hole No. 8 took 2½ in. of grout after ¾ hour. No water flowing from the hole before grouting."

August 6th, 1901.—"In the core-wall ditch we have now 13 holes grouted. To-morrow I will move the drill back and put in some intermediate holes to test the grouting. I must arrange in meantime to drain the east end of the core-wall ditch in order to uncover the sandstone, which dips rapidly into the hill. The shale, being more than 6 ft. thick, is giving considerable trouble to the drillers, so think it advisable to suspend drilling in the east end of the core-wall ditch until the sandstone is uncovered. The following is a description of grouting of holes mentioned in letter of last evening:

"Hole No. 4 W.: water standing in tube about 1 ft. high. Cylinder with grout placed and air on for 1 hour. Valve on tube closed for ¾ hour longer. Hole contained 2 ft. cement in bottom, with water above. Rammed thick mortar into tube and put on pressure again for ½ hour, 8½ in. of grout went into the hole.

Mr. Souder.

"No. 2 W.: no water flowing from this hole. Connected air hose directly to the tube, forced the water out of the crevices. Then poured in two buckets of grout (1 cement, 2 sand) after having air pressure for ½ hour, the grout had settled in the hole 2 in.

"No. 12 W.: strong stream water flowing from this hole at beginning. Forced water out, the air coming out at the sump near hole No. 1 W. Put in one bucket sand and cement dry and put on air for 10 min., after which time the hole was empty. Then put in second bucket dry grout and put on air. This stopped air from coming out at the sump. The hole was practically empty. Then put in a bucket (3 gal.) of wet grout and put on air for 15 min., which left the hole plugged. Hole No. 7: no water from this hole at beginning. Pumped water out by hand-pumps. Put in a bucket wet grout, and after pressure on 15 min. the grout had settled 2½ in."

Again, on September 9th, the writer stated:

"The core-wall ditch, where the pipe line crosses, is coming into a very hard and tough stone, massive, with little evidence so far of stratification or cleavage planes, and but little water.

"In the main core-wall ditch appearances are not so pleasing, I have had four test holes driven.

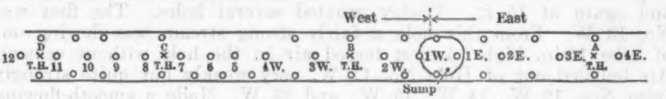


FIG. 28.

They are marked $\underset{\text{T. H.}}{\overset{\text{A}}{\times}}$ [on Fig. 28]. There was a considerable flow of water from all of them; least from B, and the strongest from C and D. Water was struck in all these holes between 11 and 16 ft. Putting air on at A, it came up at the sump, also through crevices in the bottom of the core-wall ditch near A, also in test holes, B and C. Test hole, D, was not finished at the time. Hole A took 8 buckets of grout; Hole 3 took 3 buckets.

"When air was on C air came out rather strongly at the sump, and at test hole, D, and from bottom of core-wall ditch between holes, T, and 6 W., 7 and 8 W., and 8 and 9 W. After putting in 4 buckets of grout and allowing to stand some time, cement got hard in the valve. Meanwhile we grouted test hole, D, took valve off C and found grout quite wet in C, and began to be forced out by air from D. Put long tube on C and filled with grout and applied air again on C. Then went back to D and fed it more grout, giving it in all 8¼ buckets. Air ceased to bubble up elsewhere.

"It would appear from these tests that we have a fairly tight bottom down to 10 ft. depth, the depth to which the middle holes were drilled. There is, no doubt, however, a considerable flow of water at 10 to 16 ft. depth.

"To expedite the work of drilling in part, and mainly because I think we will get better results in this work, I abandoned the old method of diamond spacing for the bore holes and started this after-

noon to drill the holes right in the middle of the core-wall ditch and spaced but 2 ft. apart. This will give us the same number of holes for a given length as previously, but, being closer, they are more likely to give us a continuous cut-off wall. At the same time, four holes can be drilled from two settings of the bar, as against three settings required for the old system." Mr. Souder.

This is shown by the sketch, Fig. 29.

"First position of bar, *A*, second, *B*. First position of drill on bar, *a*, second, *b*. Simply turn clamp over and reverse the drill in the clamp. I think this will be more satisfactory. Let me have your opinion."

To this Mr. Birkinbine replied:

"I am in receipt of your letter of yesterday, and read with interest your memoranda concerning the test holes. Remembering the fact that the first test holes were only 10 ft. deep, and we got more water in the deeper holes, your results would indicate to me that the section covering the shallow holes had probably been grouted about to their bottom, and that what you are encountering now is a practically continuous stratum below this 10 ft.

"I am inclined to believe that the appearance of air in the floor of the core-wall ditch is attributable mainly to crevices in the upper stratum, where the weight of the material is insufficient to resist the pressure. I think your plan of reversing the bar so as to drill two holes from one setting is a good one, but I am inclined to believe you can afford to separate the holes by more than 2 ft. Of course the holes from one setting will be but 2 ft. apart, but I think there could be an interval of 3 or 4 ft. between one pair, and the next pair. As to drilling these holes in the center of the core-wall ditch I should look for better results if, whenever a longitudinal crevice is evident, you drill close to this, *i. e.*, I would put the holes either to one side or the other of the core-wall ditch, according to the location of these fractures."

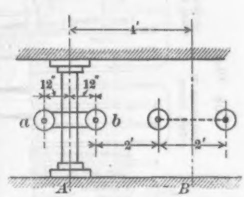


FIG. 29.

Neat cement grout was used after this, the sand being omitted. The proportion was 1 bbl. of cement to 75 gal. of grout.

At the east end three large vertical open crevices were uncovered. (Fig. 30.) The flow from these was so considerable that it was necessary to put in a force pump with a 4-in. suction and a 3-in. discharge, in order to keep the ditch clear. As the grouting method, after continued trials, did not give satisfaction at this end, it was decided to take up the bottom until the principal water strata were reached. This was done, and concrete was placed in the ditch, all the walls being plastered first with a rich cement mortar worked in

Mr. Souder.

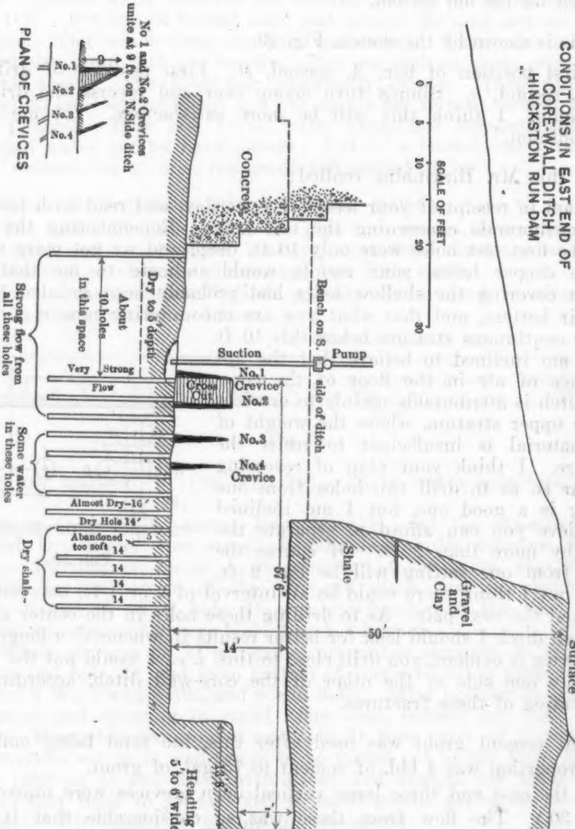
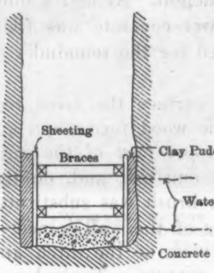
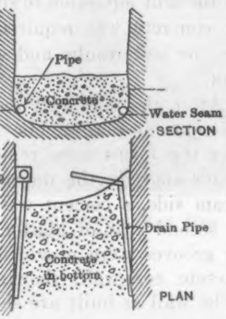


FIG. 30.

with trowels. Two coats of plaster were applied on the north or reservoir side and one on the south side. The suction pipe of the pump was built in concrete, and carried up with the wall. The strong flow of water in this section of the ditch made it difficult to place the concrete for the core-wall without having the cement washed out before it set. Mr. Souder.

At first, the method indicated in the sketch, Fig. 31, was tried, namely, a line of 1-in. sheeting was placed as shown, and clay was rammed between the sheeting and the rock to stop the water flow. Concrete was placed between the sheeting which was raised gradually as the concrete was carried up. This did not prove wholly satisfactory, and the method of piping the water directly to the sump was adopted, as shown in Fig. 32. This proved successful. Where there was too great a flow of water, plastering could not be done, but con-





METHODS OF SHUTTING OFF WATER IN THE CORE-WALL DITCH

FIG. 31. FIG. 32.

siderable neat cement was dumped in along the walls with the concrete and well worked in. After the concrete was well set, the pipes were plugged and the flow of water was shut off. As the concrete was carried up, the water came out of the ditch walls higher up, showing that the underflow had been intercepted. The core-wall ditch was extended well into the hills on each side of the valley, and test holes were bored for water, none being found, because, after a certain distance, the rock became hard and massive and free from seams. At the west end of the ditch, there was as much trouble with water. It would seem that the underflow of the whole valley was concentrated at this point, the rest of the ditch having been grouted and the flow cut off.

The drill-hole grouting method failed in the west side of the valley, as in the east, and here the bottom was also taken out, down to the water strata, and the water fought inch by inch by piping it

Mr. Souder. from the streams to the sump, as before described, and the ditch was completely filled with concrete.

The proportion for the concrete in the core-wall was 1:2:5, generally; but, at the bottom, it was much richer in cement, which was not spared in efforts to make a tight job. Near the crevices a proportion of 1:1:2½ was used. These proportions had to be varied, also, to suit the sizes of the stone supplied, which varied from ½ in. at times to 3 in. The top section of the wall was made of 1:3:6 and 1:3:7 cinder concrete. The concrete was mixed in a machine of the continuous-mixer type consisting of a long square revolving box with a helix at the back. The machine was not wholly satisfactory, but was the best to be had. The concrete was received in ½-cu. yd. dump buckets, carried on small trucks running on light track to the derricks; it was then lowered to the bottom of the ditch and dumped, as it was not thought advisable to drop it from any height. At first a middling dry concrete was required, but, later, a wet concrete was found to give the best results and was finally adopted for the remainder of the work.

After the core-wall reached above the surface the faces received two coats of plaster, one inside against the wood forms and another after the forms were removed. For the upper part of the wall, the plaster coat on the down-stream side was omitted; and, on the up-stream side, a cement wash applied with a brush was substituted for the second coat of plaster. Where sections of the wall joined, bonding grooves were provided. Fig. 33 is a view of the ditch and the concrete core-wall, showing the bonding grooves. Typical sections of the wall as built are shown on Plate XI.

The concrete core-wall contained 10 840 cu. yd., and required 13 166 bbl. of cement, or an average of 1.21 bbl. per cu. yd. The grouting and plastering took 2 078 bbl. of cement, in addition. The exact quantity of cement used in grout alone is not known.

That the work was successful was shown by the breaking out of some small springs on the inner side of the reservoir at or about the elevation of the top of the core-wall.

The comparatively low pressure of 60 lb. was insisted upon, because it was thought that extreme pressures might do harm in opening up the seams unnecessarily, and thus cause blow-outs below the dam, such as have occurred, the writer is informed, in some recent applications of the grouting method.

The result obtained by all the precautions at Hinckston Run was, not only a water-tight and practically indestructible dam, but the people in Johnstown were completely reassured, thus permitting the company to proceed with the construction of other large reservoirs in the same district.

FIG. 33.—WESTERN SECTION OF CORE-WALL DITCH, HINCKSTON RUN DAM. PARTLY COMPLETED CORE-WALL, IN BACKGROUND, WITH VERTICAL BONDING GROOVES.

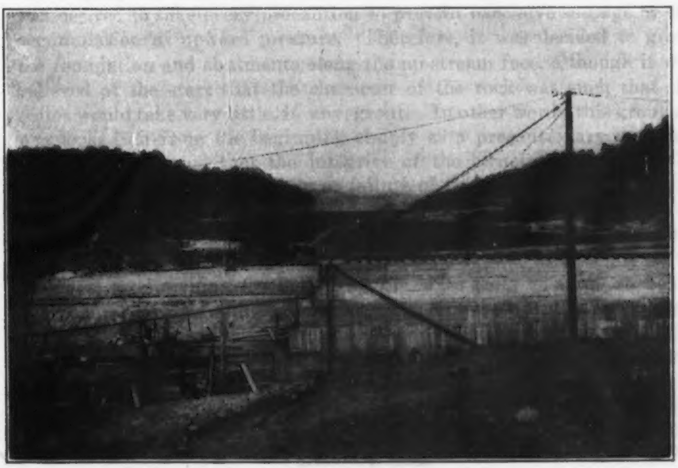


FIG. 34.—PARTLY COMPLETED CORE-WALL, HINCKSTON RUN DAM. LOOKING NORTH, TOWARD RESERVOIR.

The writer considers the grouting for the Hinckston Run Dam to have been successful. In any case, extraordinary care must be taken in applying it, and then only after conscientious study of conditions, and actual tests to determine its availability for the given locality and material. The apparent success of the grouting method for cutting off the underflow at Estacada and other places would indicate careful attention to these requirements. Mr. Souder.

The writer questions the advisability of using air pressure much in excess of 60 lb. in work of this class.

Charles H. Paul,* M. Am. Soc. C. E. (by letter).—The author has presented much interesting information, and there is no doubt that this paper, together with the discussions that it calls forth, will prove a valuable addition to engineering literature. Mr. Paul.

Foundation grouting is being carried on in connection with the construction of the Arrowrock Dam, and, although conditions there are not comparable with those at Estacada, a brief discussion of methods and results may be interesting in connection with the general consideration of this subject.

The Arrowrock Dam is a concrete masonry structure of the gravity type, 350 ft. high. The general level of the foundation in the canyon section is about 80 ft. below the river bed, and reaches a depth of 90 ft. at the deepest point. The foundation is a firm, hard granite. The rock is free from evidences of faulting, crushing, or shearing, but is characterized by rather frequent parallel joints giving it a sheeted structure which has a general north and south (across-canyon) trend, and a high dip up stream. On account of the unusual height of the dam, it was desired to take every precaution to prevent excessive seepage or the accumulation of upward pressure. Therefore, it was decided to grout the foundation and abutments along the up-stream face, although it was believed at the start that the character of the rock was such that the seams would take very little, if any, grout. In other words, this grouting was looked on from the beginning simply as a precautionary measure, and it was believed that the integrity of the structure would not be affected seriously by the success or failure of the grouting operations.

Along the up-stream face, a cut-off channel, or keyway, with a minimum depth of 10 ft., was cut out of the bed-rock, and the grout holes were drilled from the bottom of this channel to a depth of 25 or 30 ft., thus making the bottom of the holes from 35 to 40 ft. below the general level of the foundation. Two rows of holes were drilled, 5 and 13 ft., respectively, inside the line of the up-stream face of the dam, and the longitudinal distance between the holes in each row was 10 ft., with the holes in the two rows staggered.

At the beginning of the work, these holes were drilled before the rock was covered with concrete, and were brought up through the con-

* Arrowrock, Idaho.

Mr. Paul. crete, by a 3-in. wrought-iron pipe, to the elevation from which they were grouted. This pipe was set in such a way that, when grouting, the mixture could flow freely into any seam that might exist between the concrete and the rock. It was realized at the time that the most approved method for work of this nature was to drill and grout a hole before another hole was drilled in that immediate vicinity, but that plan was not followed in this case for two reasons: First, the absence of large seams and the apparent tightness of the existing seams in the bed-rock appeared to make it unnecessary to carry out the work in this manner; second, bed-rock was not uncovered until late in the fall, and as the season for placing concrete was nearly over, and it was most desirable to have the bed-rock covered with concrete before winter set in, it was necessary to push the concreting without any loss of time. Later developments proved that in this case this method of grouting was justified, as in only a very few cases, when the leakage test was applied to the hole before grouting, was there any connection between the hole tested and adjacent holes. In the few cases of this kind that did occur, a satisfactory job was obtained by grouting the two or three connecting holes in as close succession as possible, and capping the other connecting holes while one was being grouted.

During the early part of the work Burley drills were used. Later, after the foundation had been covered and the grouting operations were carried on up the abutments, pipes were set in place at the location of the holes and the holes were drilled through these pipes with a diamond drill, after a sufficient depth of concrete had been placed around them. The practice was then adopted of drilling and grouting a hole before the next one was drilled, but, on account of the tight condition of the rock, there was very little difference between the results obtained by the two methods.

The grouting machine was of the same type as that used at Estacada. The cement was a 45% blend of sand-cement (45% of granite and 55% of Portland cement), ground to such a fineness that 90% would pass the No. 200 sieve. The grout was mixed in proportions of about 1 part cement to 5 parts water, except in a few cases, when it was thickened up to about a 1:3 mixture. Pressure for the leakage test and for final application on the grout, was obtained from a tank set at an elevation of about 15 or 20 ft. above the level of full reservoir. Air for the operation of the grouting machine was obtained from the main compressor plant, and gave a pressure of about 95 lb. at the machine, which, however, was set at an elevation considerably above the bottom of the hole, so that the pressure at that point was in the neighborhood of from 150 to 160 lb. for the lowest holes.

Before making the leakage test, immediately prior to grouting, each hole was carefully washed out and sounded; then the leakage test was applied from the elevated tank. The tank was graduated to indi-

cate gallons, and the leakage test was applied to each hole, under full pressure, for a period of at least 10 min. As was expected, most of the holes showed practically no leakage before grouting, and took only about enough grout to fill the hole. In a few cases, however, an appreciable leakage was detected, and some grout was forced into the rock, although up to date the maximum quantity of cement forced into any one hole has been only about 3 bbl. To practically all the holes full air pressure was applied at the beginning, but, in a very few instances, it was found more advantageous to begin with a pressure of about 25 lb. and increase this gradually to the maximum as the hole tightened. When a hole would take no more grout, the water pressure from the tank above the dam was applied—with the idea of driving the grout home—with a pressure in excess of that which would develop from a full reservoir. At the time of this second application of water pressure, careful measurements were taken at the tank in order to make sure that the hole was tight before removing the grouting apparatus. Mr. Paul.

As to the success of the operations at Arrowrock, there is direct evidence in at least two unrelated instances that the grouting is accomplishing the results expected. At one point, near the bottom of the north abutment, where several adjacent holes showed an appreciable leakage before grouting, it was decided to drill and grout some additional holes before the pumps were pulled and the water allowed to rise to the river level. These holes were drilled with a diamond drill, from just outside the up-stream face of the dam, and were given a slight inclination, so as to bring the bottom of the hole well under the dam and in the region covered by the previous grouting operations. They were spaced so as to pass between the holes already grouted. Most of these holes were drilled to depths greater than those of the original holes, and each hole was grouted before the next one was drilled. Without exception, it was found that these holes were practically tight when the leakage test was applied, proving without question that the earlier grouting operations had tightened up the foundation successfully.

In another case, in excavating the keyway up one of the abutments, some seamy rock was encountered, which had been affected by previous grouting operations, and in the excavation of this rock, it was noted that the seams were completely filled with grout. In some cases, cracks which otherwise could hardly have been detected, could be followed by the exceedingly thin line of grout that marked them, resembling more than anything else a fine, sharp, pencil line. This grout was encountered at an elevation of at least 60 ft. above where the grouting machine had been located, showing that the grout, in this case, at least, had traveled for a long distance and had filled the seams very thoroughly.

As to the general success of grouting operations, it is believed that so much depends on local conditions that it is impossible, in any given

Mr. Paul. case, to predict just what the results may be or how far they may be relied on. It seems to the writer that a hard rock with fairly continuous seams presents ideal conditions for grouting operations, as long as the surface seams may be sealed off so that the grout may be forced down and out, rather than up to the surface, where it can escape freely. Loose gravel or sand is the other extreme, and it is very doubtful if such material can be grouted successfully. In fact, the writer has not yet heard of an instance where this has been done. All grades of material between the two may be encountered, in which grouting will give more or less satisfactory results.

The writer is of the opinion that grouting, as a precautionary measure, is excellent treatment for the foundation of any dam of considerable size, but where the safety of the structure depends on the success of the grouting operation, it would be unsafe to go ahead with construction until after grouting had been tried and proven effective. In other words, where grouting is actually needed, it is unsafe to depend on it, except after convincing evidence, at the site, that it will give the results desired.

Mr. Rands. HAROLD A. RANDS,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper has brought forth expressions and observations from members of the Profession who have had experience in grouting, and, although it was hoped that others would be heard from, it is believed that, in a general way, the discussions cover the knowledge concerning this subject at the present time. In addition to the structures cited in the paper and the discussion, in the construction of which grouting has been resorted to, mention may be made of the Hale's Bar Dam, in Tennessee, the Houlter Dam, in Montana, the Kananaskis Dam, in Alberta, and the Municipal Dam of the City of Seattle.

Possibly what might be the most interesting and instructive case of all is that of a high earth dam, the fill for which was commenced without due regard for obtaining a tight cut-off. After the dam was completed, and the pond filled, the leakage assumed serious proportions, and the back portion of the dam itself began to slough away. Holes were put down at various points through the dam and into the underlying material, and grouting was resorted to. It is reported that a total of 30 000 bbl. of cement, in the form of grout, was poured or pumped into this dam, though, unfortunately for the Profession, the "soft pedal" was put on the engineers in charge of the work, and the facts were never published.

This dam is mentioned here as showing the importance of careful attention to the blind cut-off of an earth dam. In contrast with the lack of consideration in this case, attention is called to the Lahontan Dam, in Nevada, which is also of earth, and beneath which the engineers

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* Oregon City, Ore.

of the Reclamation Service specified a concrete cut-off, 30 ft. deep, with a grouted cut-off going down an additional 30 ft., or 60 ft. in all. **Mr. Rands.**

Considering the contributions from the engineers connected with the Estacada Dam, we have first the discussion by Mr. Rippey. In the earlier paragraphs of his discussion, Mr. Rippey expresses fear lest the writer's views be given too much weight, and sums up the reasons for his somewhat extended discussion, in the following:

"In order, therefore, that the results obtained at Estacada, and the conclusions drawn therefrom by Mr. Rands, may be properly appraised by others interested in the treatment of deficient foundations, it is proper to outline, as briefly as possible, sufficient of the history of that project to indicate the unfavorable character of the governing conditions under which the work was executed."

Mr. Rippey then proceeds to attribute the comparatively poor showing made by the grouting at Estacada to "lack of co-operation" on the part of the local organization of the company owning the dam.

The writer went to considerable pains to avoid mentioning the "Chapters of Grief" which unfortunately marked the construction of the Estacada Dam, believing that the Profession at large is only concerned in such affairs when they affect results. In this particular case, it was his belief that the great amount of detailed information, together with the tables and curves submitted, constituted fairly acceptable evidence as to the care, and the regard to detail, with which the grouting was carried out.

However, since the publication of Mr. Rippey's part in the discussion, the author has received a letter from one of the foremost hydraulic engineers in the United States, wherein is expressed the opinion that grouting, to be successful, must be done with close attention and care in every detail. As this sentiment, in part, may be the result of the reading of Mr. Rippey's discussion, and possibly is shared by others, it may be proper to cover in a paragraph the history of the Estacada Dam.

During the preliminary investigation and the first three months of the construction period, the work was completely under the direction of the local organization, with Messrs. Sellers and Rippey acting as Consulting Engineers; during this period no grouting was done. During the next five months—or what may be called the middle portion of the construction period—the work was under the direction of the Sellers and Rippey organization, with Mr. William H. Cushman in charge as Resident Engineer; it was during this time that the grouting was commenced. During the remainder of the construction, the work was under the direction of the local organization, with Mr. Frank R. Fisher in charge as Resident Engineer, and with Messrs. Sellers and Rippey continuing as Consulting Engineers. The construction force, it should be said, was under the direction of the contractors, the Puget Sound

Mr. Rands. Bridge and Dredging Company, except as to the grouting, which was directed at all times and as to all details by the Resident Engineer.

The change in the middle of the work, which came as a result of conflict in organizations in a job literally loaded down with organizations, and not from anything which in any way reflects on either the integrity or engineering ability of any one, in place of affecting the grouting injuriously, really amounted to giving it a second trial, with every added care and refinement of detail which it was possible to devise. In fact, so poor was the showing being made by the grouting, both at Estacada and at the site of the proposed 150-ft. dam, at the time of the second change of management, that it is probable it would have been discontinued altogether, but for this change. Both Mr. Cushman and Mr. Fisher, it may be added, have reputations for thoroughness and attention to detail.

As to the writer's personal opinions and theories, though they might be subject to liberal discount, were they based wholly on the somewhat negative results at Estacada, they may be entitled to some additional consideration by reason of their having been tried out on another grouting job which proved more successful.

Soon after the completion of the Estacada Dam, the writer was engaged by E. G. Hopson, M. Am. Soc. C. E., Supervising Engineer of the Reclamation Service, to go to Nevada, and take charge, under D. W. Cole and F. H. Tillinghast, Members, Am. Soc. C. E., Project Engineer and Resident Engineer, respectively, of the grouting of the Lahontan Dam. This work has been described fully by Mr. Cole in articles appearing in the engineering papers, and it is interesting to contrast the results obtained in the poorer or left-channel section at Estacada with those in the poorer or second section at Lahontan.

At Estacada, the final tests showed an average leakage, at full depth, of 36 gal. per min., or 75% of the leakage of the primary holes at the same point; at Lahontan, it was only 6 gal. per min., or 9% of the leakage of the primary holes. At Estacada, a very small head of water against the coffer-dam produced, even in the final holes, a very heavy back-flow; at Lahontan, all back-flow ceased before the final holes were put down. At Estacada, communication between the seventeen final proving holes was so free that a pressure slightly exceeding the head to be produced by the dam applied to one of them caused water to flow from all the others; at Lahontan, the grouting of all the holes as soon as drilled prevented the observance of anything directly comparable to this, yet, at times, a second hole would be down to nearly full depth only 12 ft. from a hole being tested, and the fact that no water was forced from the former hole shows that there was no communication, as at Estacada.

At Lahontan, it may be stated, there was evidence of a change in the structure of the underlying material, in that in the earlier

holes put down there was much trouble from caving, and binding of the drills in the holes, in some cases causing the loss of an entire shift, and even of the bits, in getting the rods up and the caved material re-drilled; in the drilling of the final holes, no such trouble was experienced, the whole number being put down without a drill sticking or a single cave-in. **Mr. Rands.**

As the writer takes little blame to himself for the relatively poor showing at Estacada, so does he take little credit to himself for the relatively good showing made at Lahontan. Though experience taught him much during the grouting of the 34 000 ft. of drilling at Estacada, there was not enough difference between the methods used on that work and at Lahontan to account for the difference in results. The real reason, in the writer's opinion, is one of geology, and therein, and not in the lack of sympathetic co-operation, lies the vindication of Mr. Rippey.

Referring again to the paper by Mr. Schreiber, read before the Western Cement Users Association, and wherein is set forth what Mr. Rippey hoped for in recommending grouting for the Estacada Dam (page 454), it will be noted that the recommendation concludes with the following:

"It [the sufficiency of the grouting] must be susceptible of demonstration by actual test before the superstructures should be started."

At Estacada it was tried with indifferent success, due to the geological formation; at Lahontan it was tried with positive success, again due to the geological formation.

It was with no spirit of irony that the writer, in the paper, stated that the credit for evolving the grouting programme for the Estacada Dam belonged to Mr. Rippey, and it is with no spirit of irony that it is repeated. He not only proposed the programme for this work, but what is more, he gave the Profession the only safe rule to be followed in cases where grouting is to be resorted to in lieu of the usual concrete cut-off.

The proposal of Mr. Hulse to resort to slow sedimentation in lieu of the more rapid pressure grouting is a real addition to the subject, though, in the writer's opinion, this should not be resorted to until the latter had failed to produce results. The writer still believes that to continue almost indefinitely the pouring of thin cement grout into a hole is, in a great measure, to waste the cement, for, to quote Mr. White:

"It may run a long way from the part to be grouted, and on excavating this part, very little of it will be found. No one knows where it went—all that is known is that one has paid for it."

Mr. Hulse does not believe that it is safe to blast a trench in the material on which it is proposed to erect a great dam, and expresses himself as follows:

Mr. Rands. "If it [the trenching] is done by blasting, * * * there is great danger of doing almost irreparable damage to the surrounding material, and this is exactly what happened in the left channel at Estacada, where the breccia adjoining the heel-trench was shaken and cracked in all directions by the blasting."

The writer believes that this statement is far too strong, and may leave the impression that the Estacada Dam, because of the blasting for the cut-off, is standing on unsafe and insecure foundations.

It is admitted that the blasting for the cut-off trench was, in the beginning, ignorantly done; and, to support this statement, it is only necessary to say that twenty-one vertical surface holes were loaded and fired at one time. The result of this was to loosen up a large quantity of material which had to be replaced with concrete. By referring to Fig. 4, the flaring cross-section of the trench in the left channel shows the result of this method of excavation. Mr. Cushman, who arrived at the dam site before any concrete was placed, gave this matter the closest attention, had all the loosened material removed, and the bottom of the trench hand-picked to a depth of from 1 to 2 ft. below the effect of any shots. That the blasting had nothing to do with the free leakage of the holes in the left channel is readily seen by referring to the tabulation on page 467, or to the curves on Fig. 10, which indicate that the leakage, even of the primary holes, was insignificant until a depth of 30 ft. below the cut-off had been reached.

With a competent powder man in charge as foreman, there would seem to be little danger of doing irreparable damage to the surrounding rock, and against the more than one able engineer, whom Mr. Hulse states that he has heard argue against the practice, the following editorial comment* regarding the grouting at Lahontan, is quoted:

"On this dam the estimate of the saving made was only $3 000, compared with the cost of trenching and filling with concrete to the same depth. Many engineers would prefer the trenching method where the difference is so small."

In justice to the engineers of the Reclamation Service, it should be stated that it was the saving of time rather than money, which led to the adoption of the grouting programme, which, as the cut-off was already 30 ft. deep, really amounted to making assurance doubly sure, or super-insurance against leakage.

As to the general proposition of trusting to the grout to form a cut-off, even to the closing of surface seams, in lieu of the usual concrete-filled trench, there is certainly room for difference of opinion. A case in point is that of the dam at Austin, Pa.

* *Engineering News.*

The failure of this dam, in the start at least, seems to have been due to the slipping or sliding of the immediate stratum, on which the dam stood, over the stratum next below, the pressure of the water in the pond having changed the clay in the seam between the strata into an excellent lubricant. A concrete cut-off would undoubtedly have prevented the catastrophe, though it is hard to see how any amount of grouting could have altered the unfortunate result. Mr. Rands.

Mr. Fisher rather objects to the writer's expression to the effect that "most engineers will conclude that over a portion of the cut-off no grouting was needed, and that over other parts where it was needed, it did little good." This, of course, is only a matter of opinion, though it may be noted that Mr. Cushman, who is equally conversant with the Estacada conditions, considers the attempt there to have been, "to a large extent, futile."

Mr. Fisher also states that the engineers should be given credit for what was accomplished. The writer had no desire to take credit from any one, but he did feel that the Profession was entitled to the full facts concerning the Estacada grouting. Whether the grouting was carried to an unwarranted length and expense is again merely a matter of opinion; whether the method is good or bad, as applied to another structure, may be vital.

To Mr. Cushman belongs the credit for suggesting the possible shattering of the material between the drill holes in order that it might receive thick grout and form an effective cut-off, though this was not tried at Estacada, nor, so far as known, on any other work; to Mr. Cushman also is due the credit for suggesting the capitalizing of the loss through seepage and balancing it against the cost of grouting. Probably, also, more than he is aware, the writer is indebted to Mr. Cushman for ideas throughout the paper.

A member has stated it as his belief that the natural sedimentation, possibly accelerated by a deposit of suitable material above the dam, would have effectually sealed the foundation material without resorting to the grouting treatment. In reference to this, it may be of interest to consider the Crowley Creek Dam, a very thin concrete arch structure wedged between the rock walls of a box canyon in Malheur County, Oregon. This dam was designed by A. J. Wiley, M. Am. Soc. C. E., and concerning it, the following is quoted from the Third Biennial Report of the Oregon State Engineer:

"During the first season's use there was a very perceptible loss of water which passed through seams in the lava and re-appeared below the dam, but this leakage has gradually lessened and is no longer of any moment."

Considering those portions of the discussion contributed by members not connected with the Estacada Dam, there is much of value in

Mr. Rands. those of Mr. Abbot and Mr. Souder, the former covering the grouting of Lock and Dam No. 2 in the Mississippi between St. Paul and Minneapolis, and the latter that of the Hinckston Run Dam, in Pennsylvania. These seem to be the pioneer examples of grouting in America, and it is gratifying that they are now so fully made a matter of record.

In his discussion, and in his earlier excellent paper on the Fremantle Graving Dock,* Mr. Ramsbotham has given a view of grouting from another angle, and he, too, because of designing his own grouting machine, deserves to be classed among the pioneers.

Mr. White has summed up the subject so fully in accord with the writer's idea, and in so much better and so much more concise language, that to comment would be to detract.

Mr. Paul's description of the grouting of the great Arrowrock Dam, and the conclusions drawn therefrom, are valuable evidence. It is interesting to note how nearly parallel Mr. Rippey's original recommendation as to the necessity of "actual demonstration in advance of construction" is to Mr. Paul's final injunction to the effect that "where grouting is actually needed, it is unsafe to depend on it, except after convincing evidence, at the site, that it will give the results desired."

It is not proper to close this discussion without some additional statement as to the safety of the Estacada Dam. As has been said, no springs have appeared below the dam as a result of its construction, and the fact that the pressure of the water in the hole put down inside the dam was sufficient to cause the water to rise within 13 ft. of the level of the water in the pond, is good evidence that no seepage or leakage has occurred. It is true that there is the chance that the water under so great a pressure may find an outlet, but, as time goes on, and the natural silting processes take place, the danger of this becomes less likely.

The type of dam renders negligible any danger from uplift, though it may be pointed out that no seepage was encountered from the holes put down inside the dam until more than 20 ft. of rock had been drilled through, and it was not until a depth of 60 ft. of rock had been reached that the pressure became sufficient to raise the water to the height noted. The weight of this rock, plus the weight of the dam resting on the buttresses with 5-ft. footings, spaced 18 ft. from center to center, plus the resultant of the water pressure acting on the 45° deck, effectually negatived any upward pressure from the water under the dam. It is perhaps unfortunate that the term "leakage" has been applied to the water which it has been possible to "lose" through the holes in testing. For, as Mr. Fisher has pointed out, the conditions are much more favorable for leakage or loss of water, through the rock with a 50-ft. hole penetrating its interior, than when later

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1942.

the pond is filled and no such penetration exists. Also, as Mr. Fisher has pointed out, the leakage appears to be small indeed when expressed in second-feet, or rather as a fraction of a second-foot, instead of gallons per minute. Mr. Rands.

The writer feels that this explanation is due in order to counteract any inference that might be gained to the effect that, because of the failure of the grouting to do all that was hoped for it, the dam is unsafe.

In conversation, Mr. Coburn, shortly after the appearance of the paper and the first of the discussions, said that, in his opinion, the question of grouting appeared to be just where it was before, that is, it is still incumbent on those advocating grouting to do all that they claim for it to prove their point; and this seems to be the case.

Consider the dams that have either partly or entirely failed during the last few years in America. We have the Austin, Pa.; the Austin, Tex.; the Pittsfield, Mass.; the Port Angeles, Wash.; and the Stoney River, West Virginia. Of these, that at Austin, Pa., has already been considered. The dam at Austin, Tex., though, according to the late Joseph P. Frizell,* M. Am. Soc. C. E., stood on material in which "lay strata hard and soft", and though there was a single leak under the dam from the time of its completion to the day of its failure, through which flowed 6 000 000 gal. of water every 24 hours, the real cause of the failure seems to have been due to the undermining effect of the water at the down-stream toe, which, months before the dam failed, had progressed so far that bottom could not be touched with a 10-ft. pole. Had the apron been added, as recommended by the engineers, the dam in all probability would not have failed. Some may say that the lack of the apron was only one of the causes which produced the failure, and that, had some means been used to seal the material effectually along the up-stream heel, the uplift pressure would have been much reduced and the dam would not have failed at the time it did. In reply, it is only necessary to point out that grouting has not yet shown its adaptability for correcting "soft strata".

The other three dams mentioned, namely, the Pittsfield, the Port Angeles, and the Stoney River, all failed from one and the same cause—failure to carry the excavation to bed-rock. Grouting would probably not have put off their failure by so much as one day, though all might have been saved by a proper back-fill, as was suggested by Mr. Coburn for Estacada, which treatment has been most thoroughly covered in several articles by Mr. G. W. Bligh, appearing at various times in the engineering papers.†

* "Water Power," by Joseph P. Frizell, 3d ed., p. 245.

† *Engineering News*, December 29th, 1910, p. 708; February 6th, 1913, p. 226.

Mr. Rands. The real trouble with grouting is that it is too easy to think about doing; too easy, when a stratum of poor material is unexpectedly encountered in making the excavation, to set in a few pipes and go ahead with the construction, trusting to the grouting to make it right—a practice on a par with the oft-repeated expedient of putting an additional nail or two in the end of a board because of a knot or a rotten spot in the middle.

Even admitting that, because of lack of sympathetic co-operation or from any other cause, the grouting at Estacada was not done as well as it should have been, the fact that a colossal attempt, costing nearly $70 000, resulted in indifferent success, goes a long way toward proving that grouting is far from being yet a very strong competitor of the conventional cut-off. Or, considered at its best, it seems a far call from any results that it has yet achieved to the day when any material requiring treatment to carry the load of a great dam, and make it safe against the tireless, insistent, searching pressure of the water, can be rendered secure by grouting.

It is true that in the discussion mention is made of a certain foreign dam, the name and location of which the writer was not at liberty to give, wherein the grouting had been a "complete success". It is also true that those who advocate this system of foundation treatment qualify their recommendations with the provision that the sufficiency of the grouting to do the work must be demonstrated in advance of actual construction. It may be remembered, however, that only sometimes are recommendations acted on, and that even the Estacada Dam, for the benefit of which this rule was first proposed, because of the insistent demands of the owners for early power, was made an exception thereto.

The writer still believes that grouting has a legitimate field in preventing possible seepage and in solidifying the material under a solid dam in order that the uplift due to water passing under the concrete cut-off may be reduced—in short, as an agent to "make assurance doubly sure".

He regards as premature and unwarranted, by anything it has yet achieved, to claim for the grouting process the formation, in any but the most favorable material, of an effective cut-off. The real danger lies in dams of 40 to 75 ft. in height, the construction of which may be undertaken by companies of limited means, and, in the design and supervision of which, engineers of limited experience may be engaged. It is in the construction of these dams that there is danger in expecting too much of grouting, or, as Mr. White puts it, in making "a sort of fetish" of it.

The result of this, when poor material is encountered and an insistent and impatient owner is to be satisfied, will be to set a few grout casings, but the sequel may be disaster and death.

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Paper No. 1319

THE CONSTRUCTION OF THE KLONDIKE PIPE LINE*

By W. W. Edwards, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. G. B. Pillsbury, Walter S. Wheeler, and W. W. Edwards.

Synopsis.

Recent discussion of Alaska and the determination of the Government to undertake development work there has prompted the writer to submit the following paper, as describing conditions under which one job of heavy construction in the Far North was carried out. Detailed costs, he regrets to say, were not available, but wages and conditions are described and, whenever possible, day's progress figures are given.

A detailed description of methods used in construction is given, which may serve as a guide or as a comparison for other engineers engaged in laying heavy steel pipe.

The paper describes:

(1) Conditions of climate and geography peculiar to the Far North and affecting engineering work there. In particular, the perpetually frozen ground is mentioned.

(2) Details of construction.

(3) The difficulty of getting satisfactory foundations.

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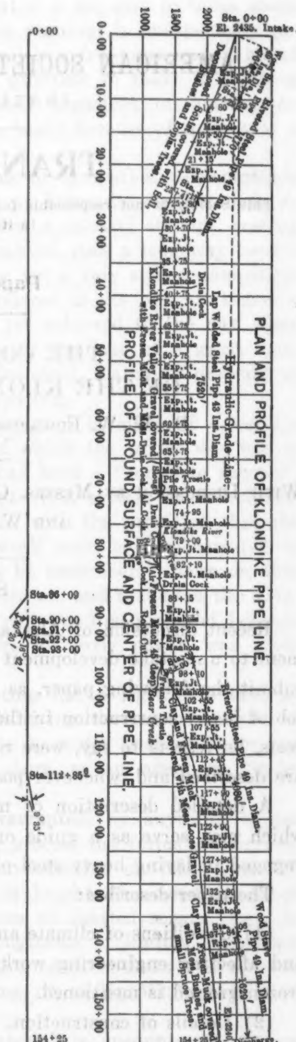
* Presented at the meeting of September 2d, 1914.

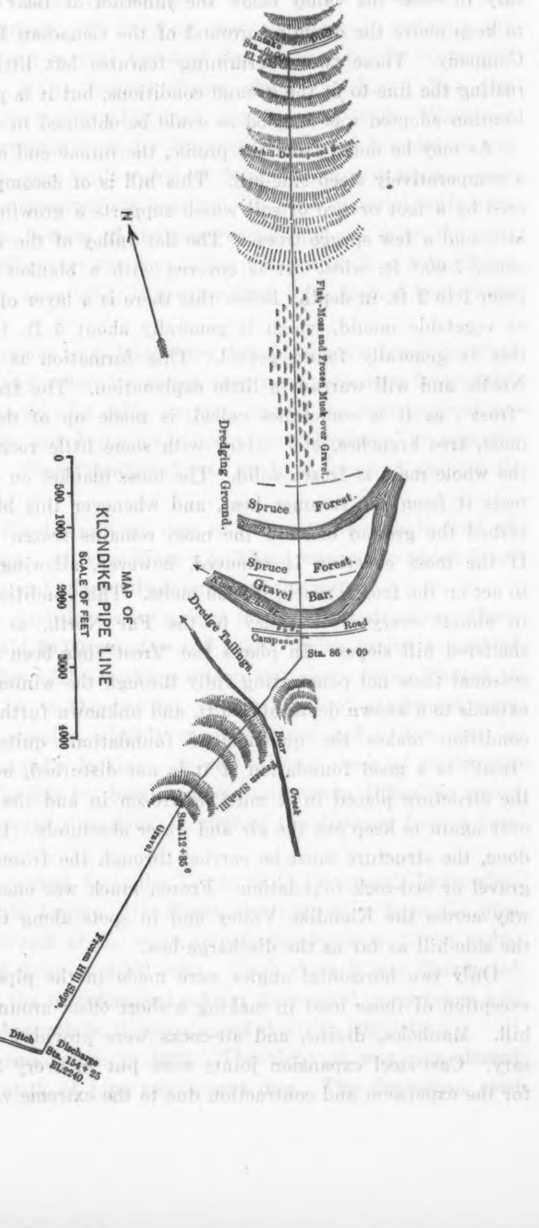
(4) Points disclosed in the test and operations of the pipe line.

The Klondike pipe line was built during 1907 and 1908 by the Yukon Gold Company, at Dawson, Yukon Territory. It is an inverted siphon crossing the Klondike River Valley, and was designed for a capacity of 125 sec-ft., which figure, however, has been slightly exceeded in operation. By reference to the profile, Fig. 1, it can be seen that the maximum head on the pipe line is about 1 100 ft. The total length of 15 760 ft. is made up of: 2 390 ft. of wood stave pipe, 49 in. inside diameter; 5 850 ft. of riveted steel pipe, 49 in. inside diameter; and 7 520 ft. of lap-welded steel pipe, 43 in. inside diameter. The steel pipe ranged in thickness from $\frac{3}{16}$ to $\frac{11}{16}$ in., and came in lengths of from 15 to 30 ft. It is the largest of several inverted siphons along the line of the Main Ditch System of the Yukon Gold Company, 70 miles in length, which supplies water for hydraulic mining operations on Bonanza Creek, near Dawson.



PLAN ON CENTER OF PIPE LINE.
FIG. 1.

The location of the pipe line was determined chiefly by two conditions: It was neces-



MAP OF KLONDIKE PIPE LINE

FIG. 2

sary to cross the valley below the junction of Bear Creek, and also to keep above the dredging ground of the Canadian Klondike Mining Company. These two determining features left little room for adjusting the line to fit the ground conditions, but it is probable that the location adopted was as good as could be obtained in any case.

As may be noted from the profile, the intake end of the pipe is on a comparatively steep side-hill. This hill is of decomposed schist covered by a foot or two of soil which supports a growth of bushes, poplars, and a few spruce trees. The flat valley of the Klondike is here about 5 000 ft. wide. It is covered with a blanket of Arctic moss from 1 to 2 ft. in depth. Below this there is a layer of frozen "muck", or vegetable mould, which is generally about 5 ft. thick, and below this is generally found gravel. This formation is peculiar to the North, and will warrant a little explanation. The frozen "muck", or "frost", as it is sometimes called, is made up of decomposed roots, moss, tree branches, etc., mixed with some little rock and sand, and the whole mass is frozen solid. The moss blanket on the surface protects it from the summer heat, and whenever this blanket is undisturbed the ground beneath the moss remains frozen the year round. If the moss covering is removed, however, allowing air and water to act on the frozen muck, it soon melts. This condition may be found in almost every river valley in the Far North, as well as on the sheltered hill slopes. In places the "frost" has been built up by the seasonal thaw not penetrating fully through the winter's freezing, and extends to a known depth of 200 ft. and unknown further depths. This condition makes the question of foundations quite serious. The "frost" is a good foundation if it is not disturbed, but, if disturbed, the structure placed in it must be frozen in and the surface mossed over again to keep out the air and water absolutely. If this cannot be done, the structure must be carried through the frozen muck to good gravel or bed-rock foundation. Frozen muck was encountered all the way across the Klondike Valley and in spots along the pipe line up the side-hill as far as the discharge box.

Only two horizontal angles were made in the pipe line, with the exception of those used in making a short offset around the base of a hill. Manholes, drains, and air-cocks were provided wherever necessary. Cast-steel expansion joints were put in every 500 ft. to allow for the expansion and contraction due to the extreme variation in tem-

perature, from 80° Fahr. in summer to — 70° Fahr. in winter. One three-span steel bridge, 295 ft. long, was necessary in crossing the Klondike River, and several hundred feet of pile trestle or framed trestle were put in at various places along the line. The aim was to have the pipe supported at least once in every 16 ft. Whenever the conditions warranted it, as for instance, when good foundation was deep, trestle bents were built every 32 ft. and king-post trusses were put in to give the intermediate bearing. In good material, sills were merely laid in the bottom of the pipe trench and on these the pipe was supported. Whenever the foundation was within 3 or 4 ft. of the pipe, crib supports were used; if greater than 4 ft., framed bents were used.

One anchor was put in at least every 500 ft., about midway between expansion joints, and, where necessary, they were used more frequently. Each anchor consisted of a cable wrapped around the pipe in the form of a clove hitch, the ends being made fast to two "deadmen" on opposite sides of the pipe. Each "deadman" was set in the ground about 6 ft. below the surface.

On the side-hills drains were dug at frequent intervals to lead off seepage and leakage. At every drain a bulkhead was put in which fitted closely to the pipe and effectually intercepted the water.

In many ways conditions were unique for heavy construction. Common labor was paid $4.00 per day and board, which meant a total of about $5.50 per day. The working season was short, being limited to 5 months—May 1st to October 1st. Some kinds of equipment were plentiful in the country, notably steam engines and boilers, but in other lines it was sadly lacking. A well selected stock of equipment was ordered for the job, but breakages and unforeseen difficulties sometimes caused awkward situations, in view of the distance from a base of supplies.

Power was furnished from the Yukon Gold Company's hydro-electric plant on Little Twelve Mile River, about 50 miles from the pipe line. It was delivered at the main transformer-house at 33 000 volts, and there stepped down to 2 300 volts. From there it was distributed at this voltage to the transformer banks at the several compressors and at the machine shop, where it was stepped down to 220 volts.

Construction was begun in 1907. The right of way was cleared and a small quantity of pipe trench was dug. The three-span steel

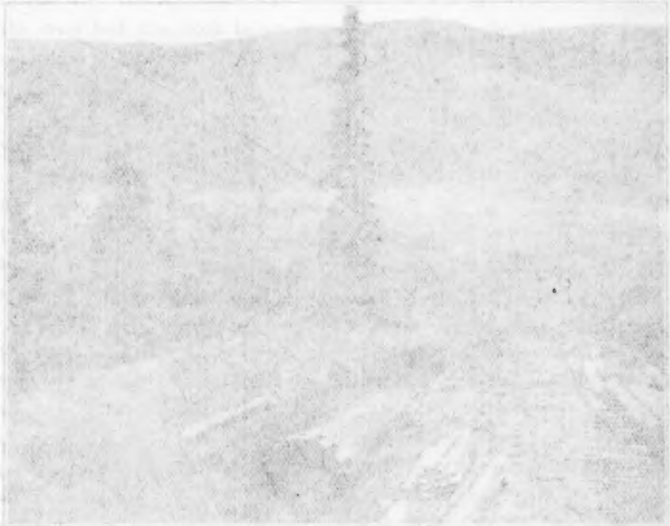
bridge over the Klondike was erected. Foundation cribs were put in the river bed, the work being done mostly while the river was frozen over and the water low. Piles were driven in the river bottom and sawed off at the gravel surface, and on them the cribs were built. These cribs were then extended to a height of about 5 ft. above high-water level and filled with rock. They were sheathed with sheet iron on the up-stream end to protect them from ice. Derricks were used in assembling the steel spans. During this season, also, a pile approach to the bridge, 400 ft. long, was built, and several other smaller pile trestles and framed trestles. Construction was halted in the midst of the working season in 1907.

In the spring of 1908 construction was begun again. It was desired to have the job completed by the following October 1st, the end of the working season. All the pipe was not then on the ground, and what was on hand was not continuous. That is, sections of pipe were missing here and there. As there were so many different kinds of pipe in the line, and as extra pieces were few, it was generally impossible to use sections of pipe interchangeably, therefore work could be started only at certain places. In the course of the work, it was necessary to start laying pipe at four different stations. An error in surveying would lead to awkward consequences where any of these headings joined, as well as at angle points in the pipe line. As there was no heavy machinery or plate for making pipe of this size in the country, and there would not be time to send "outside" for extras, it was necessary to be very careful in surveying.

All the pipe on hand was first measured up carefully and a relation established between actual measurements and those given on the blue prints. An offset line was then run paralleling the located line at a distance of 20 ft. As the seasonal thaw had loosened the original hubs so much that they could not be relied on, measurements were begun from a point on one of the bridge piers. Hubs were driven about 99 ft. apart, and measurements were taken from plumb-bob strings held from tripods. This was done on the side-hills as well as the flats. A standard pull of 15 lb. was used, the same as had been used in measuring up the pipe on hand. Corrections were made for temperature variations. Levels were run over the hubs, and horizontal distances were computed from the slope measurements and elevation differences. This system resulted in giving a first-class base line, which could be

FIG. 3.—VIEW OF INTAKE SIDE-HILL, LOOKING TOWARD DISCHARGE.

FIG. 4.—KLONDIKE RIVER AND BRIDGE, LOOKING TOWARD INTAKE.



absolutely relied on in setting grades or starting points. Grades were "shot in" by transit. Points were set along the edge of the pipe trench, and were transferred to the bottom, when needed, by using a large wooden square and a carpenter's level. Bench-marks were set every 50 ft. in elevation. Levels were determined by running ahead for half a day and then tying back to the starting point.

In construction, it was frequently found advantageous to depart from the profile in order to avoid excessive excavation, and in these cases grade changes were made by shifting angle points.

Pipe was distributed along the line by a tramway. The cars were pulled by horses across the flat and by hoists on the hills. Of these hoists, two were run by steam, and one by electricity. Signals were given by use of a wire attached to a spring pole. A telephone system also helped in signaling and general communication. Turn-tables were installed on which to change the sections of pipe end for end when necessary. Steel-frame cars were used. In hoisting, a bridle was attached to the end of the pipe, and the cable was shackled to this bridle. When the pipe was at the spot desired, the cars were chained to the rail, and the pipe, with the hoisting cable still attached, was rolled into the ditch. This held the pipe from getting away on the side-hill, and the cable always had so much spring that no harm was done to the engine.

The fitting-up gang used ratchet jacks in fitting up the pipe. The section was first slid down to the last section already connected up and then turned until it was right side up. The "front", or free end, was then raised until the top holes at the back end could be caught by drift-pins to the last section. Guide-straps were then bolted on at the joint, and the front end was lowered gradually. As the front end came down, the joint was sledged and the guide-straps were tightened as necessary. When the joint was entered all around, a ratchet turnbuckle was attached and used to pull it together. Fitting-up bolts were put in every third or fourth rivet hole. Four or five sections of pipe per 10-hour shift was an average for the fitting-up gang.

Riveting, caulking, and drilling were done with pneumatic tools. Air was furnished by 8 by 8-in. Clayton air compressors run by 15-h.p., 3-phase motors. One air compressor would furnish air enough to operate a riveting hammer, caulking hammer, drill, and air dolly, with not much to spare. The compressor outfits were mounted on

platforms and moved from place to place as desired. The riveting of the holes in the bottom of the pipe was done from the inside, and of the upper holes from the outside. Air dollies were used in holding on inside the pipe and spring dollies on the outside. Each riveting gang consisted of a riveter, bucker-up, heater, and passer. An average day's work was from 160 to 180 1-in. rivets, when conditions were good. Fitting-up bolts were used liberally, and the joint was well sledged before driving each rivet. All castings were hand-riveted to the pipe, the riveting being done on the inside.

The expansion joint castings were frequently found to be too large for the pipe. In such cases the pipe was heated by a blast lamp and belled out with hammers to fit the casting. Rivet holes in all castings were drilled in the field after assembling. All caulking was done with rounded-edge fullers, the square tool being dispensed with. The riveted pipe was caulked on the outside only, the lap-welded pipe was caulked both inside and outside. Rivets were inspected with a 1-lb. tapping hammer.

The pipe was coated on the outside with a red mineral linseed oil paint and inside with a quick-drying black paraffin paint.

The wood stave pipe was of California redwood staves thoroughly seasoned. It was laid in a trench 2 ft. deep, and, when completed, was back-filled to a depth of half the diameter of the pipe, so as to give a good bearing.

The intake penstock was equipped with overflow and turn-out, as was also the discharge box. The discharge box was located on a shaded side-hill on frozen clay. This condition made necessary considerable cribbing to protect it from the sloughing of the clay when it thawed consequent on the removal of its moss covering.

Timber was secured from adjoining leases and also from river drives. It was necessary to do considerable work to protect the river banks from undercutting in the vicinity of the bridge, as they were of loose gravel, loam, and moss. Piles were driven and shear cribs put in along the river bank above the bridge. Protection cribs were also put in above the pile bents on the bridge to take the brunt of the ice pressure and to break up the ice stream.

Frozen muck, reaching to a depth of 45 ft., was encountered between Stations 84 and 90. Shafts were dug to gravel foundation and trestle bents were put in at intervals of 32 ft. Between Stations 90 and 93

a heavy cut through dredge tailings and solid rock was necessary. This was just at the base of a hill which was covered with frozen muck. The digging of this section was delayed until as late as possible so that the extreme cold would keep the overburden on the slope above from thawing on exposure, and consequent sliding.

A small spot of frozen clay was encountered at about Station 107, and the depth of good foundation being unknown, a corduroy of spruce trees was laid on the frozen clay in the bottom of the pipe trench and this was covered with about 2 ft. of gravel. A bulkhead was built around the pipe above this place to shut off all seepage and leakage, and a drain was run off to the side. The pipe was back-filled and the surface covered with moss again. Apparently, the foundation remained frozen, as no settlement was noticed.

Although work was begun at the camp about April 1st, it was not until about May 15th that all the snow was off the ground and it was possible to work to the best advantage. A night shift was put on about May 15th and continued until October 1st, by which date the pipe was all laid. The long days of the extreme northern latitude made lights unnecessary on the work until August 1st. After that, a system of arc and incandescent lights was installed, which worked very well. The missing sections of pipe began to arrive about July 1st, and from that time onward operations were rushed. A force of 350 men was used to push the work to completion. The different headings of pipe were connected up at the expansion joints. This was effected by sliding the outside ring of the expansion joint back on the pipe and then pulling it up to position after the pipe had been lined up.

A tabulation was made showing the space to be left open at the expansion joints at various temperatures, so that they would not close during the hottest or open in the coldest weather. This was followed in setting the joints. Gauges were wired on at each expansion joint so that unequal expansion or contraction could be noted and steps taken to prevent it. The coupling bolts for each expansion joint were set so that the lap in each joint could not be less than 1 in. Reference points were also put in on each 500-ft. section of pipe, so that movement could be detected. Anchors were put in at every angle point. These were generally either bulkheads of timber against the pipe, used as thrust anchors, or cables around the pipe attached to "deadmen." When possible, at the angle points, a timber crib was put completely

over the pipe and filled with rock. The high cost of transportation and the character of the foundation made concrete anchors out of the question. The only timber available was white spruce, which was soft and brittle, and had to be used with a high factor of safety in all structures.

Some trouble was experienced from the lap-welded pipe. Two makes were used on the job. One, an American make, was perfectly satisfactory, but the other, a German make, was not. One section of the second kind cracked along the weld for a distance of 10 ft. on being dropped into the pipe trench. This was patched and reinforced, and gave no trouble in operation. Another section of this pipe burst in testing under 835 ft. head. Examination showed a flaw in the weld. When it burst, the water was shot out to the side of the pipe line and little damage was done. This was the extent of the trouble with the line in testing.

In operation, the quantity of water flowing through the pipe fluctuated considerably. As the difference in elevation between intake and discharge was 195 ft., this variation caused great fluctuations in the water level in the pipe at the intake end, there being no regulating gate at the discharge end. When little water was flowing, it ran in a very rapid stream down the bottom of the wood stave pipe at the intake end. This had the result of rapidly wearing out the bottom staves, so that considerable patching had to be done.

Bulkheads were put in at each end of the pipe and all other openings were closed as soon as the pipe was thoroughly drained. Thus the air circulation inside the pipe was shut off and the subsequent snow formed a blanket which protected it from the extremes of the winter temperature. It was found that with an air temperature of — 30 or — 40° Fahr. the temperature inside the pipe was only about — 5° Fahr. In operation, the pipe is drained at the close of the working season, about October 1st, and opened again about May 1st.

The peculiar and therefore the most interesting feature of this work was, as noted before, the study of foundations and the best styles of substructures to be used in the frozen ground. In places in the North, piles of saw-dust have been placed on the moss, and buildings put on this foundation. When water is kept away, this makes a very good foundation, as the saw-dust protects the moss and the frozen ground below from heat and consequent thawing, thus insuring

stability. On this same principle, in running a road or railroad grade across frozen ground, it is best not to cut but rather to make fills on top of the moss. Thus, on a railroad grade near Nome, a fill was made of peat cut from the surface of the tundra. When dried out and packed this gave no more trouble—until it burned up—whereas a cut in frozen ground is but an invitation to perpetual trouble from slides and settlement. In flume substructures, on the Yukon Gold Company's line, on frozen side-hills, cripple bents were tried and discarded in favor of level mudsills set in the "frost", about 2 or 3 ft. below the surface at the down-hill end. These were mossed over and remained frozen unless excessive leakage thawed the ground.

Mr. C. A. Strong, of the Yukon Gold Company, developed a type of structure which suited the foundation very well. He used piles driven into holes previously thawed out by steam points. The steam point is a strong pipe, at one end of which steam is introduced. The opposite end is rounded and a small hole left through which the steam may escape. This point is started into the frozen ground and is pounded down with a hammer as fast as it thaws through the "frost" in its way. These steam points were developed and used to considerable advantage in mining the frozen gravel. In substructure work, the steam point is used until the "frost" is thawed to the desired depth. It is then removed and a pile driven into the thawed-out hole. As the surface moss is disturbed hardly at all in this process, the pile immediately freezes in and remains in this condition. At last reports, pile substructures, such as these, were giving good service. This appears to be the cheapest form of substructure as yet devised for use in ground frozen to any considerable depth.

The writer has seen but little in technical literature dealing with construction in the Far North, and hopes that some discussion may be stimulated by this paper, especially on the subject of structures founded on frozen ground.

The foregoing work was done by day labor by the Yukon Gold Company, Mr. O. B. Perry, General Manager; C. A. Thomas, M. Am. Soc. C. E., Resident Manager; H. H. Hall, Assoc. M. Am. Soc. C. E., Superintendent of Ditch Construction, and in general charge of surveys; Mr. C. G. Newton, Superintendent of Pipe Lines; and the writer, Division Engineer and General Foreman.

DISCUSSION

Mr. Pillsbury.

G. B. Pillsbury,* M. Am. Soc. C. E. (by letter).—The description in this paper of the frozen soil in the Klondike is most interesting, but should perhaps be supplemented in order to prevent a misconception of the difficulties and possibilities of earthwork in Alaska. The writer was engaged for several years in wagon-road construction throughout the occupied portion of the Territory, and acquired a fairly intimate acquaintance with its frozen ground.

In the interior of Alaska—the area north of the enclosing coastal range—the mean annual temperature is below the freezing point. As the temperature of the ground some feet below the surface must be approximately the mean of the surface temperature, it follows that this ground is, in general, permanently frozen. The underground drainage being thus sealed, the ground-water level is high, and the country is extremely swampy. Deposits of swamp muck, therefore, are very common in the valleys and elsewhere, and they sometimes contain lenses of solid ice several feet in thickness. As the author remarks, when the moss that covers this muck is stripped, its ice content melts under the summer sun. The writer has often seen a narrow ditch quickly enlarge into a wide and deep gully through this action. It is very obvious that if ice is built on, it must be kept frozen, and in order to keep it frozen, its protecting coat of moss must be retained.

It by no means follows, however, that all the valleys are covered with muck and ice, any more than all the valleys in more temperate zones are covered with muck and water. Ordinary earth lying below the ground-water, or rather ground-ice, is frozen throughout the summer, and the difficulties of working it, with the general sloppiness of the earthwork accomplished, are similar to those encountered in work in the very early spring in the northern part of the United States. It can be loosened, however, by blasting, and the difficulties in handling are merely those common to water-saturated earth. Earth above the ground-ice level thaws and dries readily, and its working is entirely normal.

The writer has traveled over a railroad near Nome to which the author possibly refers in the latter part of his paper. It was a very light surface road, and had the misfortune to be located, for the greater part of its length, on a frozen swamp. Light ditches had been dug through the moss and turf on each side, and these had gullied out so badly that much of the track had to be shifted to one side. The journey of about 60 miles occupied an entire day, although the passengers escaped the usual upset. The writer, however, does not think that any moral can be drawn from this railroad; a swamp does not

* New London, Conn.

Mr. Pillsbury.

afford the best location for a railroad in any country, and this road was merely fortunate enough to find its swamp frozen.

The prediction is ventured that good railroad construction in Alaska will follow standard practice closely, henceforth as well as heretofore. Swamps will be crossed on a fill, as usual. The principal departure from construction elsewhere will be in the matter of unit costs, and these, on account of the short working season and distance from supplies, will be several times greater than on any road in the United States.

Mr. Wheeler.

WALTER S. WHEELER,* M. AM. SOC. C. E. (by letter).—Records at Skagway, Alaska, show that during the open season of 1907, the Guggenheims shipped into Dawson more than \$5 000 000 worth of freight, a considerable portion of which was material for the Klondike pipe line of the Yukon Gold Company. The freight was landed, from ocean steamships, at the wharf in Skagway, and transhipped, over the White Pass and Yukon Narrow-Gauge Railroad, 111 miles to White Horse, and then by steamer down the Yukon to Dawson. The freight and passenger rates over this road are naturally quite high, as a considerable portion is built through exceedingly mountainous country, and the bulk of travel is confined to the summer, when navigation is open on the Yukon River. The track has a 3.9% grade for about 16 of the first 20 miles out of Skagway, to the famous White Pass, which is about 2 800 ft. above Skagway and mean tide. This portion of the road cost \$2 000 000, or an average of \$100 000 per mile, and it is said that the road more than paid for itself during the first two years of operation.

Mixed trains leaving Skagway are drawn by four engines distributed equally throughout the same. At a point 8 miles from Skagway, a rock thrown into the river from the track struck the water in 5 sec.

By the "Far North", the writer assumes that the author includes all of Alaska and the adjoining Yukon Territory, and also British Columbia lying directly east of Alaska.

Costs for material, labor, etc., vary considerably for different locations. At the time the Klondike pipe line was constructed, with common labor at \$4 per day and board, the Alaska Road Commission was paying \$2.50 per day and board on the trail out from Haines Mission, with a working season of 5 months in each location. Machinists were paid \$5 per day in Skagway, with extra for overtime, and \$1 per hour in Dawson, Yukon Territory, and the adjoining creeks. Cord-wood, the principal item of fuel, cost \$5.50 per cord delivered in Skagway, and \$15 per cord in Fairbanks, Alaska. Board and room, the best obtainable, could be had in Skagway for \$40 per month,

* New York City.

Mr. Wheeler. while the same class of board and room cost $75 per month in St. Michael, Alaska, and Dawson, Yukon Territory, and $125 in Fairbanks.

The cost of electric light in Skagway was 10 cents per kw-hr., and in Fairbanks, 51 cents. Business telephones cost $5 per month in Skagway, and $20 in Dawson and Fairbanks.

These prices were current about 1907 and 1908, and though some of them have changed from time to time, the writer has given them in order to show that they vary considerably in different locations of the "Far North", and that estimates for construction in the "Interior" are, as a rule, much higher than along the Coast.

A fair grade of coal could be purchased in Skagway for $13 per ton, but, in St. Michael, the writer has paid $25 for coal of the same grade. This coal was all shipped from the United States or from lower British Columbia, while billions of tons of the best bituminous and anthracite are waiting to be mined along the coast of Southern Alaska as soon as the Government sees fit to allow it to be opened up.

At Skagway the temperature seldom falls below zero, but it is not unusual to experience a temperature of —70° Fahr. in midwinter at Dawson, Fairbanks, and Nome, and sometimes —80° Fahr. When the temperature falls below about —40° Fahr., there is never any wind, and, the air being very dry at that temperature, the real danger to life is from exposure when it appears to be warmer than it really is.

In summer, ordinary vegetables are grown in abundance, and wild raspberries, blueberries, cranberries, and currants are plentiful.

An excellent breeding place for mosquitoes is formed below the niggerheads (bunches of coarse wild grass) on the tundra, and during May, June, and July, it is impossible to work on the tundras without the protection of nets and gloves; but August and September are comparatively free from these pests, which are not fever-bearing in the North.

It is impossible to make foundations to any great extent with the peat taken from the tundras, as it will dry out and burn up in the numerous tundra fires during the summer, as mentioned by the author in the case of the Nome Railroad.

In places near fresh and salt water, where suction dredges are available, good foundations can be made by running the discharge pipe so that the material will be deposited on the tundra, provided the water is not so rough as to prevent the dredge from operating. The writer had charge of dredging the St. Michael Canal during the summers of 1908 and 1909, and the material discharged from the pipe line would have made excellent foundations on the tundra, if they had been desired. When carrying on construction near the water, it is well to quarter the men on houseboats anchored well out from shore, so that, while not on duty, they can get some respite from mosquitoes. In this way one can obtain a more efficient working crew.

W. W. EDWARDS,* Assoc. M. Am. Soc. C. E. (by letter).—The writer urges that no attempt be made to build a structure, or to grade, on frozen ground, if there is any practicable way to avoid it. In the case of the Klondike pipe line nothing else could be done. The same condition has been observed in many other instances. Building on "frost" is at best a makeshift, and one never has the feeling that he has done a good job. A structure founded on frozen ground will probably need jacking up into shape once in a while, and a railroad over frozen ground will need considerable work by the maintenance-of-way gang with tamping and line bars. Mr. Edwards.

Probably the most trying work which the writer has ever observed is ditching in frozen ground. When this is attempted, the right of way should be cleared of trees, brush, and moss, at least a year before the excavation is to be made, so that the sun's heat will thaw the ground to the required depth. In ground which is not too steep or does not contain too many ice stringers, this method may result in a good ditch; but where the ground surface has a considerable slope, the ditch excavation generally starts slides from above, which cause much trouble.

Then, again, the finished ditch may appear to be all right, but, when water is turned in, it will percolate until it reaches an ice stringer along which it follows until it comes out to the ground surface below. In this way a ditch may break in the bottom, and the water may come out to the surface 100 ft. or more below the grade, leaving the ditch bank intact. Repairing such breaks requires brush, moss, soil, and much skill and patience in puddling. Ditches in such material are generally made wide and shallow, the lower bank being protected on the inside with strips of moss.

The writer does not wish to give the impression that all the soil in Alaska and Yukon Territory is frozen the year round; but spots of frozen ground are frequently encountered, especially in the Yukon Basin, and, as in the case of the Klondike pipe line, often cannot be avoided economically.

* San José, Cal.

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Paper No. 1320

HUACAL DAM, SONORA, MEXICO*.

By H. Hawgood, M. Am. Soc. C. E.

With Discussion by Messrs. Charles W. Sherman, M. M. O'Shaughnessy, L. R. Jorgensen, L. J. Mensch, and H. Hawgood.

Synopsis.

This paper describes the methods used in the design and construction of a concrete arch dam dependent on arch action for its stability. The dimensions of the dam are: crest length of curved structure, 140 ft.; radius, 76 ft.; maximum height, 100 ft.

The futility of attempting a determination of stresses by pure mathematical processes is discussed. It is pointed out that the lack of definite knowledge as to the behavior, under stress, of the rock in which the ends and base of the dam are intrenched, and to which it is joined, inhibits the application of pure mathematics, and the use of the simple formula for a thin cylinder is sustained.

For general information, and for comparative purposes, Table 1 gives the characteristic dimensions and stresses of numerous pure arch dams in various parts of the world.

The cost of the work is given, and there is also a description of the pipe line, conveying the impounded water from the dam to the place of use, together with its cost.

The works were executed in a difficult country, with impossible wagon roads, where the cement, sand, and some other materials had to be packed on animals, and where labor was disturbed by a state of war.

* Presented at the meeting of May 20th, 1914.

The concrete dam described in this paper is an arched structure dependent on arch action for its stability. The use of the arch for dams and dependence on it for stability, is old; how old is not susceptible of proof, but certainly more than a century. Some time prior to 1800 was built the Meer Allum Dam, at Hyderabad, India, a multiple-arch structure composed of a succession of brick arches, varying from 40 to 82 ft. in radius, with an aggregate length of about ½ mile and a water depth of about 40 ft. A more recent Indian example is a dam 60 ft. high, also of the multiple-arch type, at Alwar, Rajputana. In Europe the Zola Dam, near Aix, France, of a radius of 158 ft. and a height of about 120 ft., dates from 1843.

In more recent years many arch dams have been built, notably in New South Wales and South Australia. Their characteristic dimensions are given in Table 1.

In the United States, the Bear Valley Dam, Southern California, built in 1886, has in the past attracted much attention by the boldness of its design, and yet its audacity is equalled by the less known Upper Otay Dam of the San Diego Water Supply, built in 1900. The granite masonry of the Bear Valley Dam was subjected to compressive stresses of about 61 tons per sq. ft., and the concrete of the Upper Otay Dam is subjected to stresses of similar intensity. The Otay Dam is an instructive illustration of the stresses which can be sustained successfully by a horizontal arch of Portland cement concrete, mixed 1:2.1:3.4, when founded and abutting on strong, hard rock. The advisability, however, of imposing such high stresses may well be questioned.

It is of interest to note that the old Bear Valley Dam, after doing duty for 27 years, has at last survived its usefulness. It has been superseded, and at high-water stages is submerged by the water impounded by a higher dam of the inclined multiple-arch type, designed and built by J. S. Eastwood,* M. Am. Soc. C. E.

The writer has not found any record of the failure of a true arch dam; if any have failed, it would be well to have all the facts brought to light.

It will be noted that the stresses sustained by the various structures have an extremely wide range, varying from a minimum of 10.6 tons to a maximum of 61 tons per sq. ft., or from 147 to 870 lb. per sq. in.

* *Engineering News*, December 25th, 1913, Vol. 70, p. 1284.

TABLE 1.—CHARACTERISTICS

Date of construction.	Location or name.	R, Radius, in feet.	L, Top length of segment, in feet.	Arc, in degrees.
UNITED STATES OF AMERICA.				
a 1886	Bear Valley, Cal	335	300	51° 20′
b } *c* } 1900	Upper Otay, Cal	359	350	56° 00′
d 1907	Crowley Creek, Ore	70	150	55° 50′
e 1913	"Goodwin Dam," Stanislaus River, Cal. Twin arches, each	135	238	100° 00′
MEXICO.				
1912	Huacal, Sonora	76	140	105° 40′
SOUTH AUSTRALIA.				
1904	Barrosa	200	472.5	135° 20′
NEW SOUTH WALES.				
1897	Parkes	300	540	100° 00′
1898	Costamundra	250	475	109° 30′
1898	Tamworth	250	440	100° 00′
1899	Wellington	150	350	133° 00′
1899	Mudgee	258	498	112° 00′
1899	Wollongong	200	344	98° 30′
1896	Lithgow No. 1	100	162	93° 00′
1897	Picton	120	112	58° 00′
1898	Queen Charlotte Vale	90	113	72° 00′
1858 } 1898 }	Paramatta	160	225	80° 00′
1906	Lithgow No. 2	100	221	127° 00′
1906	Medlow	60	81	78° 00′
1907	Barren Jack	80	....	....
INDIA.				
g Prior to 1800	Meer Allum	82	178	175° 00′
FRANCE.				
h 1843	Zola, Aix	158	205	74° 20′

AUTHORITY.

a. Bear Valley Company's Original Records, also Schuyler's "Reservoirs for Irrigation, Waterpower and Domestic Supply," pp. 246-256.
b. *Engineering News*, Vol. 46, p. 125.
c. *Engineering News*, Vol. 51, p. 337.
d. *Engineering News*, Vol. 66, p. 220.
e. *Engineering News*, Vol. 70, p. 748.

After study of the available local materials and labor conditions, a stress of 16.3 tons per sq. ft., 226 lb. per sq. in., was adopted for the Huacal Dam. If labor experienced in concrete work had been available at Nacozari, a higher stress might have been considered. The most vigorous supervision cannot entirely eliminate the errors of uninformed

OF CONSTRUCTED "ARCH-DAMS".

D, Depth of water, in feet.	*T*, Thickness at *D*, in feet.	*S*, Stress, in tons per square foot.	Nature of aggregate.	Class of masonry.
48.0	8.42	61.0	Granitic.	Granite masonry.
75.0	14.00	61.0	Granitic.	Concrete.
50.0	12.00	46.7	Granitic.	Concrete.
63.0	5.16	26.6	Lava.	Concrete.
70.0	at *D*= 61.0 ft. 16.0 "	21.4	Gravel.	Concrete.
88.5	12.83	16.3	Andesitic.	Concrete.
94.0	34.00	17.3	Gneiss.	Concrete.
37.0	13.50	29.1	Granitic.	Concrete.
45.5	12.88	28.2	Granitic.	Concrete.
62.0	21.50	23.3	Granitic.	Concrete.
48.5	10.00	23.7	Conglomerate.	Concrete.
50.0	18.00	22.4	Slate.	Concrete.
41.5	11.62	22.9	Basalt.	Concrete.
38.0	11.38	11.4	Sandstone.	Concrete.
40.0	13.62	13.8	Sandstone.	Conerete.
34.0	8.65	11.7	Quartzite.	Concrete.
46.0	15.00	15.3	Sandstone.	Concrete.
78.5	24.00	10.6	Sandstone.	Concrete.
66.0	8.96	14.4	Sandstone.	Concrete.
38.0	5.00	19.0	....	Concrete.
39.0	8.50	11.8	....	Brick masonry.
119.7	41.80	14.2	....	Rubble masonry.

f. *Minutes of Proceedings*, Inst. C. E., Vol. 178.
g *Minutes of Proceedings*, Inst. C. E., Vol. 172, p. 214.
h Schuyler's "Reservoirs for Irrigation, Waterpower and Domestic Supply," p. 362.

The stresses, *S*, in Table 1, are computed by the cylinder formula:

$$\text{Stress, in tons (2 000 lb.) per sq. ft.} = \frac{\text{Radius (feet)} \times \text{Depth (feet)}}{32 \times \text{Thickness (feet)}}.$$

labor; foremen, however competent, cannot be everywhere at once, and it is prudent to make allowance for probable inferior quality of the labor, an important factor in the proper placing of concrete.

The physical characteristics of the coarse aggregate enter largely into the question of allowable compressive stresses.

GENERAL.

The Huacal Dam was constructed during 1911-12 for the Moctezuma Copper Company, Nacozari, Sonora, Mexico (Phelps, Dodge and Company, New York), from the designs and under the general direction of the writer.

The purpose of the dam is to impound storm waters and create a gravity supply for the milling operations of the Copper Company. The water supply prior to the construction of the dam was obtained by pumping from the gravel beds of the Rio Nacozari. For a period of from 2 to 4 months of each year the waters of these gravels became so far depleted as to impose limitations on the output of the mill. The normal capacity of the mill is 400 tons of concentrates per day, and the required daily supply of water is about 1 250 000 gal. The pumping plant, now out of service, but held as an emergency reserve, consists of two 10 by 12-in. Aldrich triplex plunger pumps, electrically driven through gearing, discharging against a head of 225 ft. In round figures, the yearly cost of operating these pumps was $12 500. Decrease in cost and increase in water supply constituted the economic incentive for the dam.

Nacozari, the present southern terminus of the Ferro Carril de Nacozari, is 76.6 miles south of the International Boundary. The northern termini of the railroad are Douglas, Ariz., on the American side of the line, and Agua Prieta, on the Mexican side.

The dam is built across the canyon of Huacal Creek, about 3 miles northeast from Nacozari, at an elevation of 550 ft. above the town and 4 100 ft. above sea level. Nacozari formed the base of construction supplies. Bulky pieces were transported by wagon, over a rough mountain road some 6 miles in length, with steep gradients, the maximum load being 1 ton to four good animals. Everything else, including all cement and sand, was packed on mules and burros, with loads of 300 lb. or more to the mule, and 175 lb. to the burro. The average cost of packing was about $1.02 per 2 000 lb. The cement was packed from the railroad depot, a distance of about 6 miles, and the sand from the Nacozari River, about 4 miles. The cost per ton-mile, one way, averaged about 21¾ cents.

All prices and costs are given in United States gold values.

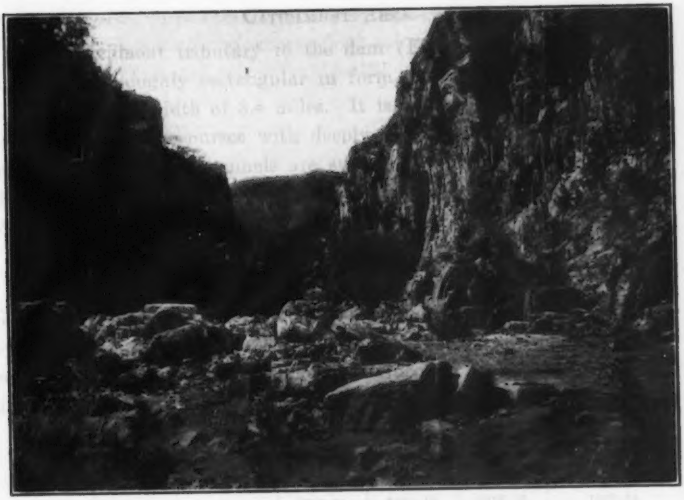
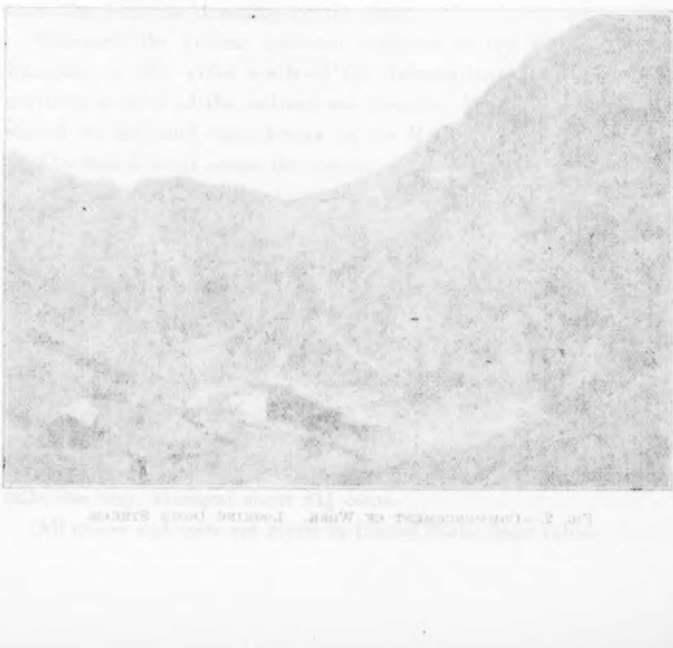


FIG. 1.—DAM SITE, LOOKING DOWN STREAM.

FIG. 2.—COMMENCEMENT OF WORK. LOOKING DOWN STREAM.



CATCHMENT AREA.

The catchment tributary to the dam (Fig. 3) has an area of 13 sq. miles, is roughly rectangular in form, with an extreme length of 5.7 miles and a width of 3.4 miles. It is a mountainous area, broken by numerous watercourses with deeply incised channels. The slopes leading down to the channels are steep, vegetation is sparse, the soil shallow, with much bare rock, and the storm discharges are of a torrential character.

RAINFALL.

The rains commence in June. May is a dry month, and the rain records are in the best form for comparative purposes when arranged by seasons running from June 1st to May 31st of the succeeding year. A rain gauge has been kept at Nacozari since May, 1910, but no definite information as to previous rainfall can be obtained.

TABLE 2.—RAINFALL AT NACOZARI BY SEASONS.

Month.	1910-11. Inches.	1911-12. Inches.	1912-13. Inches.	1913-14. Inches.
June	0.51	2.66	1.17	0.08
July	4.39	3.73	4.66	4.13
August	2.32	2.25	3.59	2.77
September	0.89	4.79	0.00	
October	0.14	3.36	0.54	
November	0.33	1.38	0.83	
December	0.00	1.13	0.77	Highest rate of precipitation 3.05 in. in 3 hours, October 4th, 1911.
January	1.73	0.00	0.84	
February	1.43	0.31	4.52	
March	0.01	1.96	0.57	
April	0.60	0.20	0.53	
May	0.00	0.00	0.02	
Totals	12.35	21.77	18.04	....

No gauges are maintained other than the one at Nacozari. The rainfall over the catchment area above the dam is unquestionably greater than at Nacozari, but to what extent is as yet purely a matter of estimate and conjecture.

RUN-OFF.

On the basis of the probable rainfall and the topographical features of the catchment area, it was estimated that the yield of a rain season such as 1910-11 would be approximately 3 000 acre-ft. (2 700 000 gal. per day). The season of 1910-11 was classed as "dry" by those familiar with the country. This opinion is supported by the rain records of

1911-12, 1912-13, as far as they go, but they are of too short duration to form any conclusions, and it cannot be assumed that there will not be dryer seasons than 1910-11. The margin between the con-

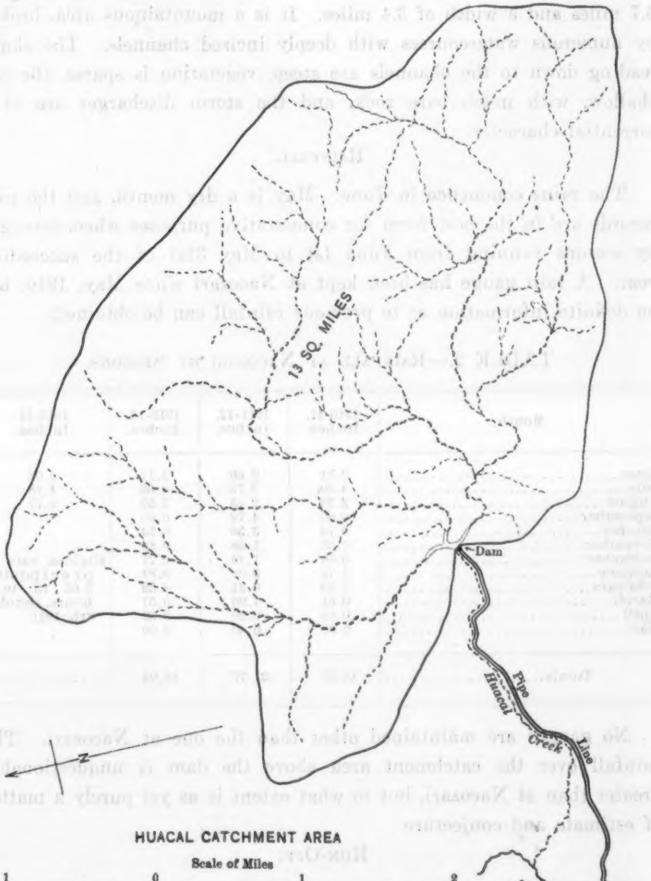


FIG. 3.

templated use of 1 400 acre-ft. annually, or 1 250 000 gal. daily, and the 3 000 acre-ft., or 2 700 000 gal. daily run-off estimated for 1910-11 is considered ample to take care of dryer seasons.

On August 14th, 1910, a storm run-off was observed lasting about 1 hour, the total discharge from which was estimated at 300 acre-ft., with an average run-off rate of 255 sec-ft. per sq. mile.

On October 5th, 1911, a flood, having a maximum rate of 2 000 cu. ft. per sec., passed, without damage, over the partly finished dam, then 40 ft. high. The total discharge during this storm was estimated to be 1 240 acre-ft., with a maximum run-off rate of 154 sec-ft. per sq. mile.

Reservoir and Dam Site.

In outline, the reservoir is T-shaped, as shown by Fig. 4. The dam and outlet of the reservoir are on the stem of the T, to each side of which the water surface at high water extends for about 1 mile.

The capacity of the reservoir at the crest level of the dam is 2 700 acre-ft., with a surface area of 93 acres. It is designed, should it be found desirable to impound more water, to install tumbling flash-boards along the crest of the dam.

At the point selected for the dam the canyon had in its natural state a bottom width of from 20 to 30 ft., and, at 90 ft. above the bottom, a width of 150 ft.

The geological formation is Andesitic. In the main, the rock is hard and close-grained. The dip of the formation approaches the vertical, and the strike is across the canyon approximately at right angles to its axis, both of which are conditions favorable to minimum leakage and stability of foundation. A seam discharging a little water was disclosed in driving the outlet tunnel and in the shaft beneath the outlet tower. During the active progress of the work it was impracticable to isolate and measure this water. In April, 1912, after the water in the reservoir had risen against the dam to a height of about 40 ft., this spring was measured and found to yield 1.5 gal. per min. The entire flow over the rock bed of the stream at a point about 100 ft. below the dam was, on the same date, 3.58 gal. per min. This would ascribe a seepage through the dam and its foundation contact of 2.1 gal. per min. In February, 1913, 10 months later, the entire flow over the stream bed was again measured and found to have decreased to 2.83 gal. per min., the depth of water against the dam having in the interim increased from 40 to 60 ft.

The seepage through the dam and its foundation contact was not precisely segregated, but was judged not to exceed 1.0 or 1.5 gal. per

On August 14th, 1910, a storm run-off was observed lasting about 1 hour, the total discharge from which was estimated at 800 acre-ft., with an average run-off rate of [illegible] sec-ft. per sq. mile.

On October 8th, 1911, a flood, having a maximum rate of 2 000 cu. ft. per sec., passed, without damage, over the partly finished dam, then 40 ft. high. The total discharge during this storm was estimated to be 1 240 acre-ft., with a maximum [illegible] rate of 1.4 sec-ft. per sq. mile.

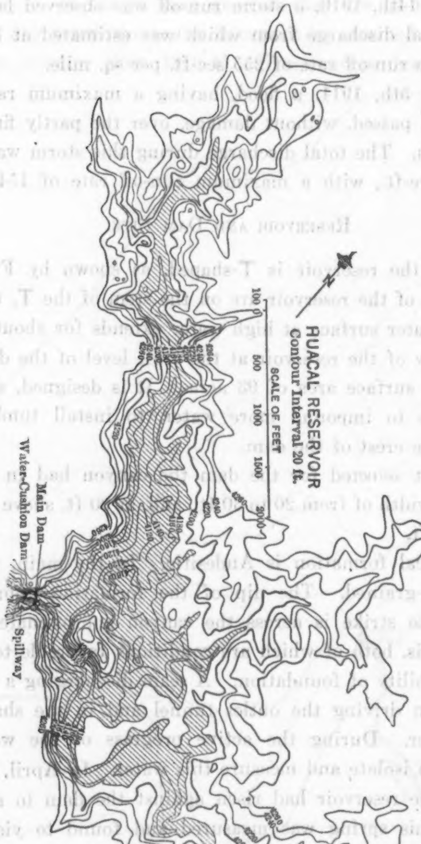


FIG. 4.

RESERVOIR AND [illegible]

In outline, the reservoir is T-shaped, as shown by Fig. 4. The dam and outlet of the reservoir are on the stem of the T, to each side of which the water surface at high water extends for about 1 mile.

The capacity of the reservoir at the crest level of the dam is 2 700 acre-ft., with a surface area of 88 acres. It was designed, should it be found desirable to impound more water, to install tumbling flash-boards along the crest of the dam.

At the point selected for the dam the canyon had in its natural state a bottom width of from 20 to 30 ft., and [illegible] 80 ft. above the bottom a width of 150 ft.

The geological formation is Andesite [illegible]; the rock is hard and close-grained. The dip of the strata approaches the vertical, and the strike is across the canyon approximately at right angles to its axis, both of which are favorable conditions to minimum leakage and stability of foundation. [illegible] a little water was disclosed in sinking the outlet tunnel [illegible] the shaft beneath the outlet tower. During the earlier stages of the work it was impracticable to isolate and measure this water, but in April, 1912, after the water in the reservoir had risen against the dam to a height of about 40 ft., this spring was measured and found to yield 1.5 gal. per min. The entire flow over the [illegible] bed of the stream at a point about 100 ft. below the dam was, on the same date, 3.58 gal. per min. This would [illegible] seepage through the dam and its foundation contact of 2.1 gal. per min. In February, 1913, 10 months later, the entire flow over the stream bed was again measured and found to have decreased to 2.58 gal. per min., the depth of water against the dam having in the interim increased from 40 to 60 ft.

The seepage through the dam and its foundation contact was not precisely segregated, but was judged not to exceed 1.0 or 1.5 gal. per

min. This trivial seepage speaks well for the care taken in making and placing the concrete, and in obtaining sound contact between the concrete and the native rock.

DESIGN OF DAM.

The narrowness of the canyon and the character of its rock sides and floor suggested an arch dam (Fig. 5) as the economic type for the location. Comparative studies with a rock-fill dam as a practical alternative confirmed the selection of the arch type.

The requisite cross-section of the dam was determined by the simple formula for cylinders subjected to external pressure:

$$\left.\begin{array}{l}\text{Stress, in tons}\\ \text{(2 000 lb.) per sq. ft.}\end{array}\right\} = \frac{\text{Radius (feet)} \times \text{Pressure (pounds per square foot)}}{\text{Thickness (feet)}}.$$

The values finally adopted in the design of the dam were: Stress, 16.3 tons per sq. ft.; radius, 76 ft.; and the equation for thickness became $T = \frac{76}{16.3} \times \frac{D}{32}$, whence the thickness, T, at the depth, $D = 0.146\ D$.

With these values the theoretical triangular dam section has a vertical front face, and a back face on a batter of 1¾ in. to the foot. The lower portion of the dam is built of this form, the upper portion being widened to meet the practical necessities which govern a dam top.

The formula does not satisfy all the conditions, mathematically, for the dam is neither a complete cylinder with free ends, nor is it everywhere free to expand and contract; on the contrary, it is a segment connected at its bottom and sides with the immovable, but not the less elastic, country rock; and it is not a thin cylinder, but one of appreciable thickness. The problem is impossible of exact mathematical solution, for, however approached, its treatment is based on assumptions. The break of continuity in both form and material at the line of contact between the artificial structure and the natural rock, and the unknown value of the element of deformation in the natural rock, create uncertainties which, in themselves, render a mathematical demonstration based solely on absolute facts an impossibility, for the facts are not and cannot be known. Such elements of uncertainty are not confined to arch dams alone, they are equally prevalent with gravity dams. The researches and experiments of Pearson, Otley,

Brightmore, Baker, and others have shown the accepted middle-third method of gravity section design to fall short of being absolutely correct. Nevertheless, the fact remains that dams designed on that theory,

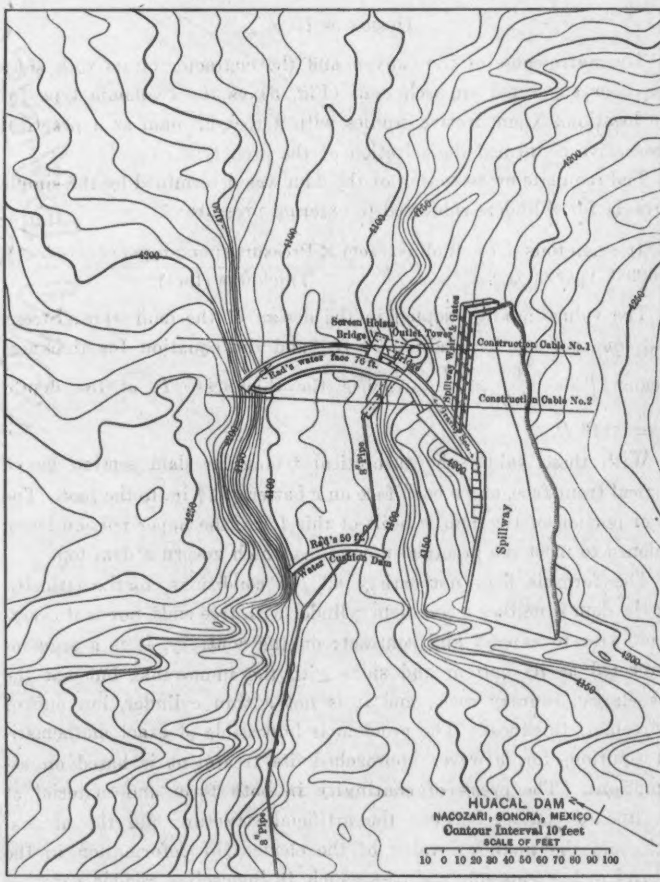


FIG. 5.

within certain recognized limitations as to intensity of stress, have been and continue to be successful. Similarly, the many Australian arch dams designed on the simple cylinder theory have been and continue to be successful.

Whatever the unknown and undeterminable stresses may be, particularly those around the foundations, the adopted formulas, empirical though they may be, have given and do give safe results, and, in the present status of knowledge as to the actual stresses in dams, their use must be considered to be more conservative than that of formulas mathematically correct but based on assumptions of unknown conditions. The successful practice of engineering is one of applied science, rather than of pure science, and the pure mathematical formula, without some modifying coefficient derived from practice, is a rarity.

The views expressed by Dr. W. C. Unwin, Past-President of the Institution of Civil Engineers, in a discussion on the subject of stresses in dams, are very pertinent:*

"It had now been shown that it was hopeless to attack the dam problem by pure mathematics. He did not think there was any theory on which an engineer relied in designing a bridge, a roof, or a retaining wall, to which mathematical objections could not be raised of the kind that had been raised against the ordinary theory of dams. The state of stress in a plate round a rivet was as complex as that in the foundation of a dam, and was as little susceptible of mathematical treatment; and yet engineers were not afraid of making riveted joints."

It may be said, with equal truth, that engineers will not be afraid to continue building gravity dams on the middle-third theory, or arch dams on the cylinder theory, and no disaster will follow, always provided that there is due recognition of the conditions of the foundation and abutments which Nature provides.

The futility of attempting a pure mathematical determination of the stresses in a dam is well exemplified by the measurements of the actual deflections of an arch dam at Barren Jack, New South Wales.†

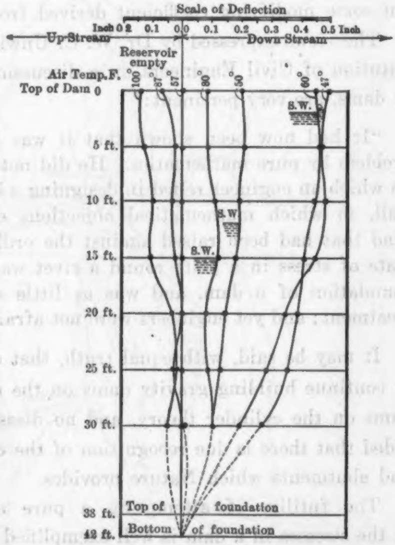
The dimensions of this dam are given in Table 1. The structure is of concrete with a light vertical reinforcement (against temperature stresses) of 20-lb. steel rails, 10 ft. from center to center and 1 ft. from the face. As the steel area is only 0.044% of the area of the concrete at the base, and 0.139% at the top of the dam, its presence is negligible, as far as bending stresses are concerned.

The deflections of the Barren Jack Dam, which are shown in the diagram, Fig. 6, were taken under varying conditions of tempera-

* *Minutes of Proceedings*, Inst. C. E., Vol. 172, p. 156.

† *Minutes of Proceedings*, Inst. C. E., Vol. 178, p. 61.

ture and water depth. The observations made when the reservoir was empty and the air temperature 57°, which approximates the closest of the observations to mean temperature, are drawn as a straight line, and the other observations are plotted in regard thereto. The curves drawn through the various observed points are brought for convenience to one common point of origin on the foundation base. This is not strictly correct, for it makes the impossible assumption that the base is absolutely rigid, and that the rock on which the dam stands is incapable of elastic deformation under the stresses transferred to it by the dam and by the weight of water supported by the rock bed immediately above the dam. Deformation of the rock undoubtedly takes place, and, such being the case, the arch action takes place in some unknown degree from the very base of the dam. That deformation of the bed-rock does take place to considerable depths was the opinion expressed by the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in discussing the Coolgardie Water Supply.*



DEFLECTIONS OF BARREN JACK ARCH DAM
At different temperatures and heights of water.
Measured at 5-ft. vertical intervals from top of dam to 30-ft. below
Minutes of Proceedings, Inst. C. E. Vol. 178

FIG. 6.

Fig. 6 indicates that the bending stresses at the base are probably little if any greater than at higher points of the dam. Were the bending stresses at the bases of arch dams as great as some theories would lead us to think, many if not all the arch dams cited would have been fractured, and in particular the very thin dams of Bear Valley and the Upper Otay. That they have not been fractured is conclusive that the theories need amending.

* *Minutes of Proceedings*, Inst. C. E., Vol. 162, p. 123.

The extreme height of the Huacal Dam from base to crest is 100.25 ft. The foundation trench was excavated until sound rock was reached at depths from 5 to 15 ft. That there might be no powder cracks, hand tools only were used in the bottom of the trench. Similar precaution was exercised in the abutment trenches at each end of the dam, which were carried horizontally into the rock formation to a maximum distance of 20 ft. to insure contact with sound rock.

The front face of the dam is vertical, the back face has a variable batter changing from 1¾ in. to the foot at the bottom, to ½ in. to the foot at the top, as shown on the cross-section, Fig. 7. As actually built, the back face does not conform with this design precisely. By a mishap, the dam about its middle height is somewhat thicker than designed, as shown by the dotted line on the cross-section. The profile was returned to the true lines with an easy batter, as shown in the drawing.

The top of the arched portion of the dam is made hollow, both to save concrete and to provide free intercommunication between the space beneath the sheet of falling water when the dam is overflowed, and the open air, and thus avoid the tremors incidental to the making and breaking of contact between water and dam with the forming and breaking of a vacuum behind the nappe when the free passage of air is wholly or partly suppressed.

The dam is not of the arch form for its entire length. For 42 ft. at its southeasterly end it is of gravity section, and in direction tangential to the curve.

The continuance of the dam on a curve throughout its entire length would have brought the direction of its easterly end too closely parallel to the axis of the canyon to have been economical or safe.

In the canyon below the main structure there is a supplementary dam designed to create at times of overflow a water cushion with a maximum depth of 20 ft. over the base of the main dam.

Spillway.

No stream discharge records being available, the determination of spillway capacity necessary for safety became largely speculative and matter of opinion. The subject was studied in conjunction with D. C. Henny, M. Am. Soc. C. E., and after weighing all available data it was concluded that provision to pass a flood of 6 000 sec-ft. would

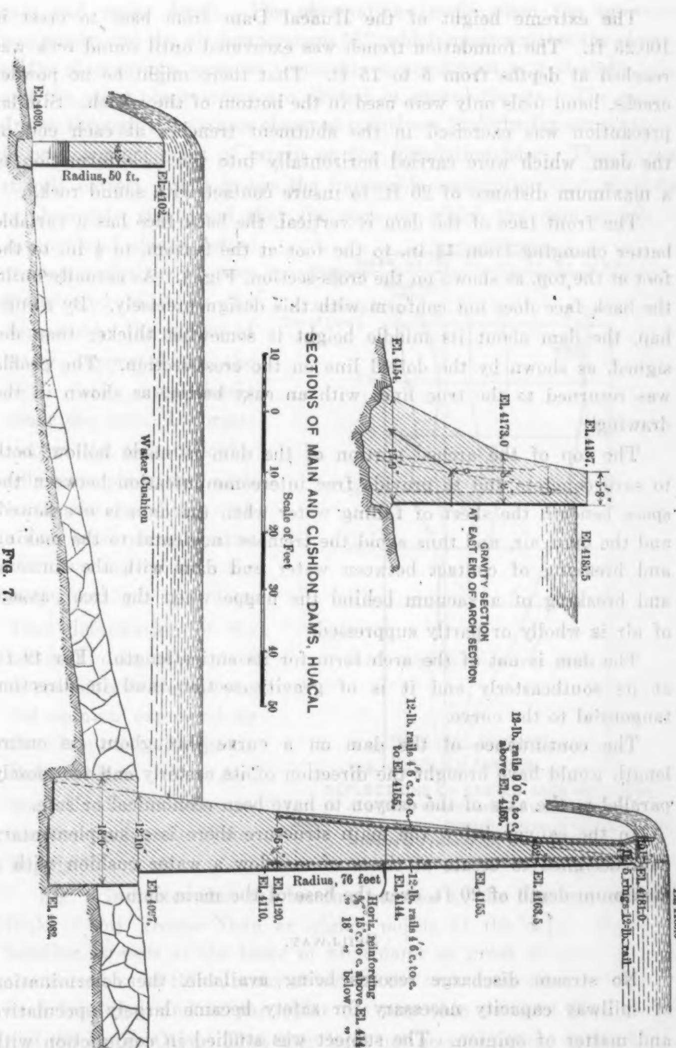

SECTIONS OF MAIN AND CUSHION DAMS, HUACAL
Scale of Feet
10 0 10 20 30 40 50
GRAVITY SECTION
AT EAST END OF ARCH SECTION
El. 4187.
5′8″
El. 4185.5
El. 4173.0
5′2″
El. 4154.
17′9″
El. 4185.5
El. 4181.0
5 rings 12-lb. rail
El. 4163.5
12-lb. rails 9′0″c. to c. above El. 4165.
El. 4155.
12-lb. rails 4′6″c. to c. to El. 4155.
12-lb. rails 4′6″c. to c.
El. 4144.
Horiz. reinforcing
⅝″ 15″c. to c. above El. 4144.
" 18" " below "
Radius, 76 feet
9′5½″
El. 4120.
El. 4110.
El. 4097.
12′10″
18′0″
El. 4082.
Water Cushion
El. 4102.
4′2″
Radius, 50 ft.
El. 4080.

FIG. 7.

FIG. 8.—A QUIET DAY. LABORERS SCARCE.

FIG. 9.—A BUSY DAY.

be ample. If this quantity is ever attained, 2 600 sec-ft. would flow over the top of the dam, and 3 400 sec-ft. through the spillway. Provision for a run-off of 6 000 sec-ft. from a catchment area of 13 sq. miles, or 452 sec-ft. per sq. mile, may be excessive, and there is some local evidence that such is the case; on the other hand, flood peaks of greater volume are not unknown in the semi-arid districts. The writer had occasion to make an exhaustive investigation of the Chase Creek, Arizona, flood of December, 1906. Chase Creek, a tributary of the Gila River, is about 200 miles north of Nacozari. Both localities have somewhat similar climates. The peak of the Chase Creek flood had a discharge rate of 647 sec-ft. per sq. mile from an area of 20 miles. The duration of the flood wave was about 40 min., with a mean rate of flow during that period of about 420 sec-ft. per sq. mile. The slopes of the country draining into Chase Creek are precipitous, averaging 1 100 ft. fall to the mile. The drainage slopes of the Huacal Basin are much flatter, and the channels less direct, consequently, the maximum discharge rate for the same rainfall will be less from the Huacal water-shed than from one of such abrupt topography as Chase Creek. Provision for 452 sec-ft. on the Huacal is probably more than the equivalent of 647 sec-ft. on Chase Creek.

The spillway discharge channel receives its water through nine openings regulated by pivot-gates. The aggregate effective length of the openings is 67.5 ft. These openings are separated from one another by 7-in. reinforced concrete partitions set at an angle of 45° with the axis of the spillway sill wall, and making an angle of 27° with the line of flow through the spillway discharge channel. Abrupt deflections of stream lines are thus avoided, and the capacity of the discharge channel is correspondingly enhanced.

The spillway gates hang on vertical pivots, which are slightly to one side of the center of the gate. The greater water pressure on one leaf of the gate than on the other, to the extent of the pivotal eccentricity, is used to make the gates self-closing and to keep them closed as long as the water level is below the danger mark. The automatic opening of the gates is accomplished by carrying the shorter leaf to a greater height than the longer one. A rise of water above the lower leaf will reverse the previous balance of pressure, and the gates will open automatically.

Outlet System.

The discharge of water from the reservoir is effected by pipes controlled by flap-valves outside and standard screw-valves inside a concrete outlet tower, Fig. 10. The intake ends of the pipes are protected against trash by screens movable for cleaning.

The short bridge connecting the top of the tower with the dam is not connected to it rigidly. This leaves the tower free for independent movement in the event of an earthquake.

The gate-tower is a continuation of a shaft in the rock formation, from the bottom of which a pipe tunnel, in rock, runs out under the dam. The pipes consist of an 8-in. service pipe, a 12-in. blow-off, and a 10-in. drain. The tunnel is refilled with concrete in which the 12-in. and 10-in. pipes are embedded, together with an additional 10-in. pipe through which runs the 8-in. service pipe. This arrangement makes the service pipe removable for repairs or renewal. Should the other pipes rust out, the holes left through the concrete will answer all purposes.

The lower 8-in. intake pipe and the 12-in. blow-off are carried through a short tunnel into the tower-shaft. These pipes are of $\frac{3}{4}$-in. cast iron, embedded in the concrete of the tower. All these pipes are provided with the usual collars or flanges to stop seepage.

Cement and Sand.

The cement used was furnished, under the Standard Specifications of the American Society of Civil Engineers for Portland Cement, by the Southwestern Portland Cement Company, El Paso, Tex.

Tests at the dam, of neat cement, and cement and sand briquettes, gave the minima shown in Table 3.

The appearance and feel of the Huacal sand were not reassuring, but as the use of the local sand, if possible, meant a material reduction in cost, and as the cement and sand briquette tests promised well, a trial of the Huacal sand was made in a block of concrete containing some 80 cu. yd. The results were anything but satisfactory. After 10 days of setting a pick could be driven 1 in. into the mass without any great muscular energy. The total mass was removed, and the hope of using local sand was abandoned.

Good sand is always more or less scarce in arid regions, a fact largely attributable to insufficient natural grinding and washing to destroy and remove the softer particles. This is particularly true of locations such

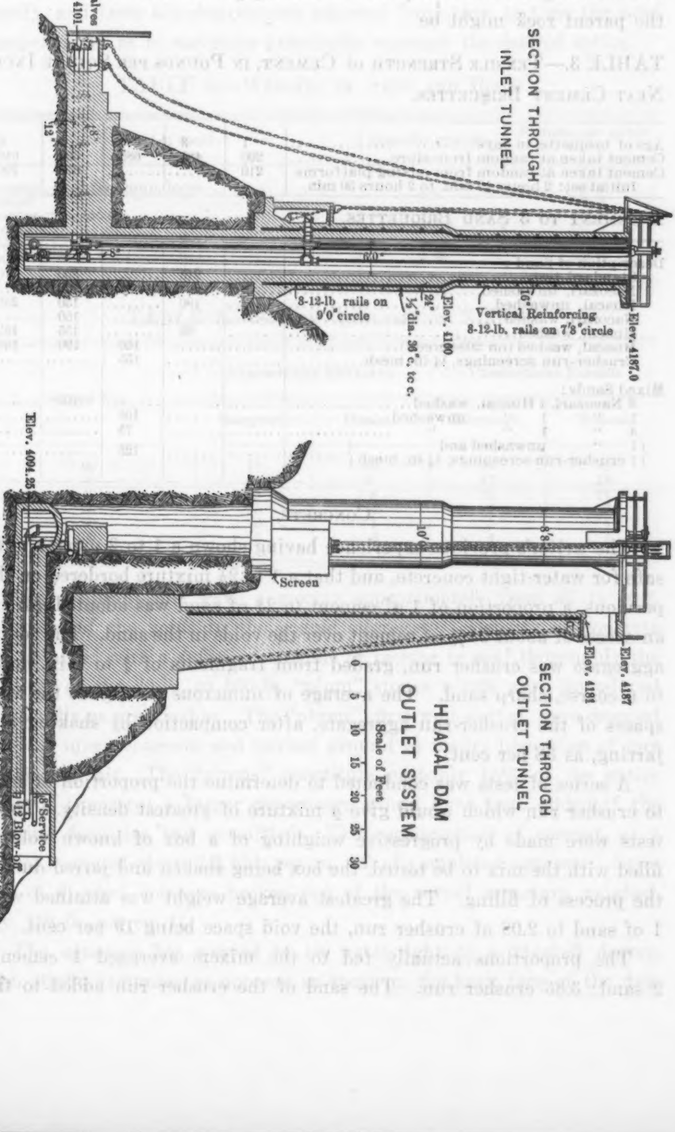

SECTION THROUGH
INLET TUNNEL
Flap-Valves
Elev. 4101
Elev. 4187.0
Vertical Reinforcing
8-12-lb. rails on 7'8" circle
Elev. 4160
8-12-lb rails on
9'0" circle
Screen
Elev. 4094.25
SECTION THROUGH
OUTLET TUNNEL
Elev. 4187
Elev. 4181
HUACAL DAM
OUTLET SYSTEM
0 5 10 15 20 25 30
Scale of Feet
8" Service
12" Blow-off
10" Drain

FIG. 10.

as the one in question, where the greatest length from the dam site to the basin crest is 5 miles, a distance far too short to expose the rock detritus, in its journey along the various stream beds, to the scouring and abrading influences requisite to making good sand, however suitable the parent rock might be.

TABLE 3.—TENSILE STRENGTH OF CEMENT, IN POUNDS PER SQUARE INCH.

NEAT CEMENT BRIQUETTES.

Age of briquette, in days	1	3	4	5	6
Cement taken at random from store	290	485	685	660	640
Cement taken at random from mixing platforms.	210	....	....	675	790
Initial set: 2 hours 10 min. to 2 hours 30 min.					

1 CEMENT TO 3 SAND BRIQUETTES.

Description of Sand :					
Standard test	....	215	....	300	310
Nacozari, unwashed	....	....	88	100	....
Huacal, unwashed	....	180	....	180	235
Nacozari, washed	....	....	....	150	....
Huacal, washed	....	95	....	155	165
Huacal, washed (on 200-screen)	....	....	160	190	150
Crusher-run screenings, ¼-in. mesh	....	....	155	....	....
Mixed Sands :					
3 Nacozari, 1 Huacal, washed	....	....	....	100	....
1 " 1 " unwashed	....	....	106	....	....
4 " 1 " "	....	....	75	....	....
1 " unwashed and 1 crusher-run screenings, ¼-in. mesh	....	....	125	....	....

CONCRETE.

The writer's previous experience having shown a 1 to 2 mortar to be safe for water-tight concrete, and that a 1 to $2\frac{1}{2}$ mixture bordered on the pervious, a proportion of 1 of cement to $2\frac{1}{4}$ of sand was adopted, giving an excess of about 25% of cement over the voids in the sand. The coarse aggregate was crusher run, graded from fragments of $\frac{3}{4}$ to 1 in. down to a coarse, sharp sand. The average of numerous tests gave the void spaces of the crusher-run aggregate, after compaction by shaking and jarring, as 38 per cent.

A series of tests was conducted to determine the proportion of sand to crusher run which would give a mixture of greatest density. These tests were made by progressive weighing of a box of known volume filled with the mix to be tested, the box being shaken and jarred during the process of filling. The greatest average weight was attained with 1 of sand to 2.03 of crusher run, the void space being 19 per cent.

The proportions actually fed to the mixers averaged 1 cement: 2 sand: 3.85 crusher run. The sand of the crusher run added to the

straight sand made the desired 1 to 2¼ mortar. The proportion of sand in the crusher run varied, of course, from time to time, with the wear and changing and consequent slacking and tightening of the crusher jaws. The foreman watched the composition of the crusher run closely, and from his observations adjusted from time to time the sand component so as to maintain practically constant the desired ratios.

TABLE 4.—WEIGHTS OF SAND AND ROCK.

Kind of sand.	Specific gravity.	Pounds per cubic foot.
Nacozari, slightly damp, loose	....	105.2
Huacal " " "	....	101.8
Crusher-run, loose	....	104.8
Nacozari	2.51	157.0
Huacal	2.43	152.0
Andesitic rock fed to crusher	2.65	166.0

TABLE 5.—SCREEN ANALYSIS OF SAND.

Screen No.	Percentage Retained.		Percentage Passed.	
	Nacozari.	Huacal.	Nacozari.	Huacal.
20	71	60	29	40
30	12	21	17	19
40	5	8	12	11
60	5	4	7	7
80	2	1	5	6
100	1	2	4	4

The resultant mix gave a mortar approximately from 35 to 40% in excess of the voids in the coarse aggregate. Density of concrete was obtained with a sufficient surplus of mortar to coat thoroughly the surfaces of the 20-lb. to 300-lb. "plum" stones which were introduced as liberally as practicable. The "plums", first well wetted, were dropped into the mush concrete and worked around to insure liberation of any imprisoned air. The "plums" constitute probably 15% of the entire mass of concrete; a larger percentage would have been added if the hoisting facilities had permitted. The consumption of cement, as a whole, averaged about 1.3 bbl. per cu. yd. of completed concrete. Samples of finished concrete broken out of the actual structure weighed, dry, 152 lb. per cu. ft.

The concrete has proved to be water-tight to a marked degree, such slight dampness as appears in spots on the back face of the dam

being more probably due to seepage at over-night joints between one day's work and another than to percolation through the body of the concrete.

Plant.

The concrete plant was erected on the east side of the canyon at a higher elevation than the top of the dam. Cement was unloaded directly from the pack animals into store at the highest point of the plant, and the transportation was paid for by count of the sacks delivered. Sand, with transportation paid for by delivered weight, was unloaded and weighed into a bin near the bottom of the canyon and hoisted by bucket on inclined cable and delivered into gravity bins. The rock crusher was operated at an elevation corresponding with the spillway floor, and fed by rock from the spillway excavation. The product of the crusher was raised by belt and bucket elevator to the bins.

Cement, sand, and crushed rock gravitated from their respective bins to the batch-measuring hoppers, and thence to two mixers, one with a capacity of 12.7 cu. ft. to the batch, and the other of 21.7 cu. ft. The average composition of the mix was 1 cement: 2 sand: 3.85 crusher run.

The mixers delivered into a 1½-cu. yd. bucket slung between two standing cables, one crossing the canyon immediately below the dam and the other immediately above. Traveling carriages, one on each cable, moved in and out together. Hoisting tackle, independently operated, reached from each traveling carriage to the bucket bail. This arrangement permitted the depositing at will of the bucket at any point on the dam. Traveling and hoisting ropes were operated at speeds of about 600 ft. per min. light and 400 ft. per min. loaded. The maximum quantity of concrete placed in any one day was 114 cu. yd., and the average per day for the days on which the plant was running was 37.5 cu. yd. The minimum working time of mixing and placing 1 cu. yd. of concrete was 3.2 min., the maximum was 19.25 min. per cu. yd. (refilling outlet tunnels), with an average working time of 8.7 min. per cu. yd. for the whole work.

The machinery, picked up around the company's mines, and not without some incursions into the scrap pile, was of a heterogeneous character, incompatible with compactness or efficiency of operation. A saving in plant investment was effected at the expense of the constructional cost of the dam.

There were five steam boilers, ranging from 10 to 35 h.p., supplied with feed-water by a direct-acting, duplex, steam pump, drawing from water impounded above the dam. Six engines were used, a 25-h.p. for hoisting and a 30-h.p. for traversing the carriages on the cableways, a 26-h.p. on the rock crusher, a 30-h.p. on the sand hoist, and two 6-h.p. engines driving the concrete mixers. The fuel supply was chiefly wood from the live oaks cleared from the reservoir, liberally helped out with coal to compensate for the wood being green.

It would have been feasible to operate by electric current transmitted from the copper company's generating plant at Nacozari. This method was contemplated originally, but at the time was not deemed best. The experience of the work showed that the elimination of boiler and enginemen troubles and the elimination of transportation of coal, and other economies which would have followed with electricity as the motive power, would have resulted in a lower cost of construction.

The work was carried on during the Madero Revolution, when the country was in a state of turmoil; near-by collisions between the opposing forces were frequent, and the labor supply was disturbed and uncertain.

Forms.

The passing of any bolts, wire, or other form fastening, through from face to face of the dam was prohibited. The lower portion of the dam was constructed with wood forms of the customary stud and sheeting type, to a height above which the external struts bracing the studding would have become inconveniently long. The upper 60 ft. of the dam was built with steel forms (Fig. 11). These were in units 9 ft. wide for the water face and 8 ft. 3 in. for the back face, and 12 ft. high. They were generally used in threes, making a block space for pouring concrete 27 by 12 ft. and of the width of the dam.

No attempt was made to vary the radius of the back face forms to conform to increasing radius due to decreasing thickness of dam with height. The forms were shaped to a compromise radius. The slight angle made by the adjacent forms, where not strictly conforming to the true radius, was imperceptible on the finished work. The location of the dam and its commercial use did not warrant the increased cost of forms of variable radius which would have been necessary for precision of shape. The front face of the dam being vertical,

There were five steam boilers, ranging from 10 to 35 h.p., supplied with feed-water by a direct-acting, duplex, steam pump drawing from water impounded [illegible] the dam. Six engines were used, a 30-h.p. for hoisting and a 30-h.p. for traversing the carriages on the cableway, a [illegible]-h.p. on the rock crusher, a 20-h.p. on the sand hoist, and two [illegible] engines driving the concrete mixers. The fuel supply was [illegible] wood from the live oak secured from the reservoir, [illegible] with [illegible] to compensate for the wood being green.

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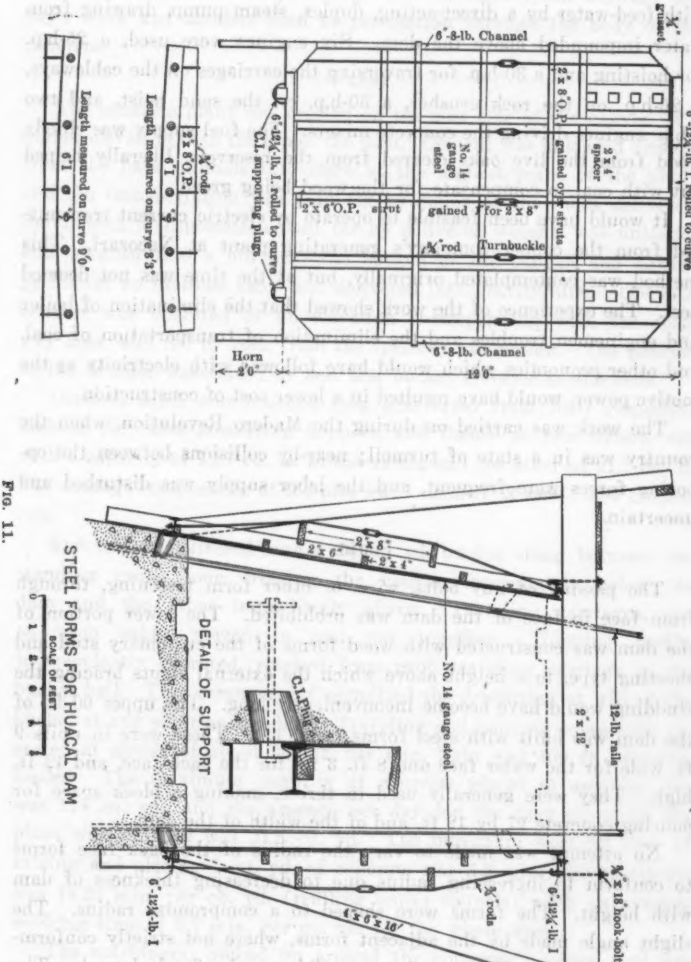


FIG. 11. STEEL FORMS FOR HUACAL DAM

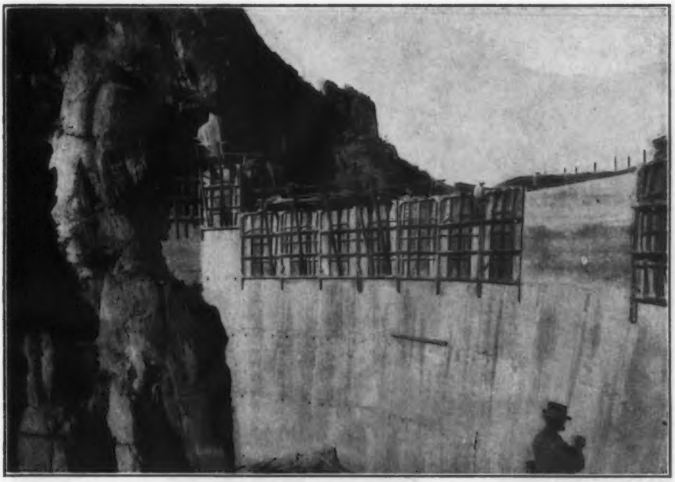


FIG. 12.—REAR FACE OF DAM, SHOWING STEEL FORMS IN USE.

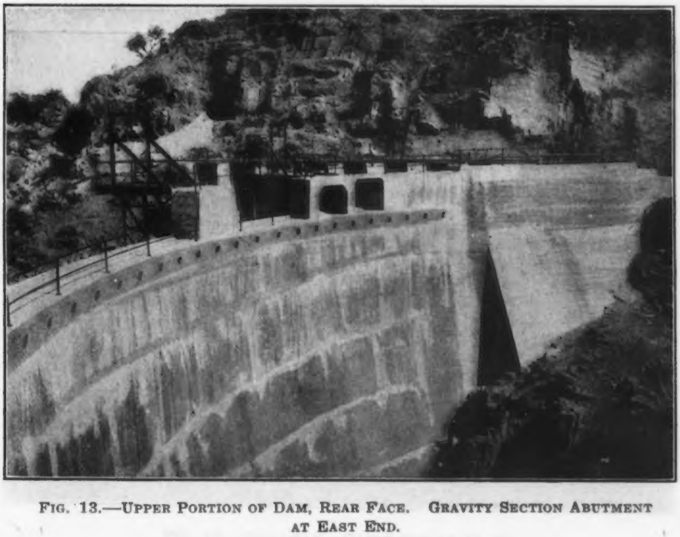


FIG. 13.—UPPER PORTION OF DAM, REAR FACE. GRAVITY SECTION ABUTMENT AT EAST END.

Fig. 14.—Top of Dam, from Spillway.

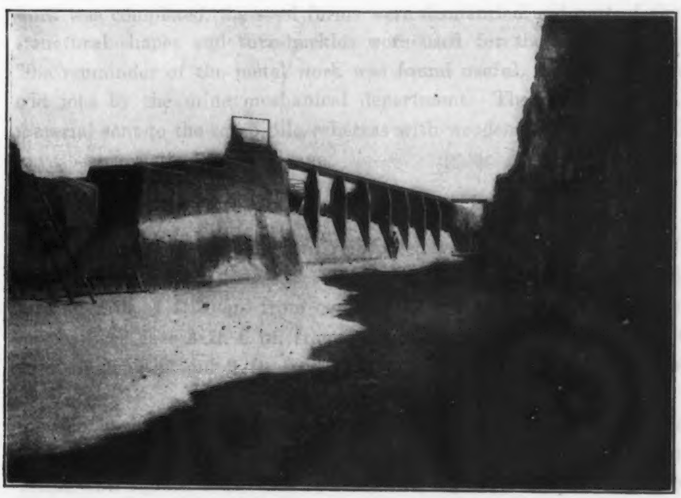


Fig. 15.—Spillway. Gates not in Place.

FIG. 14.—TOP OF DAM, FROM SPILLWAY.

FIG. 15.—SPILLWAY. GATES NOT IN PLACE.

its radius is constant. The front forms, being first set accurately to position, constituted a base from which were set the back forms.

The steel forms were supported by their lower edges engaging in hook-shaped projections on taper cast-iron plugs built in advance into the concrete as the work progressed. The plugs retained their position by virtue of being inclined downward into the concrete. After use, the plugs were withdrawn to be used again and the pockets were filled with concrete. Two plugs were used to a form unit, one under each corner. The plan proved satisfactory, except in some instances where the concrete bucket with its load struck heavily the top of the form and jarred the plugs loose. However, no instance occurred where the plugs actually let go. A bolt might be added to the plug, as shown by the broken lines on Fig. 11, which could be removed with the plug, the washer alone being lost in the concrete.

The walkway on top of the forms fulfilled admirably its purpose of a safe and convenient working platform for the concrete gang.

The forms were handled into place by the overhead cableways. The cost and time of removing the steel forms from set concrete and placing them in new position was low and in marked contrast to the cost and time of stripping the ordinary wooden forms. After the concrete work was completed, the steel forms were dismantled and part of their structural shapes and turn-buckles were used for the spillway gates. The remainder of the metal work was found useful, and absorbed for odd jobs by the mine mechanical department. There was but little material sent to the scrap pile, whereas with wooden forms there would have been practically no salvage.

Metal Reinforcements.

Both faces of the dam are reinforced vertically and horizontally against temperature stresses. The vertical members are old 12-lb. mining rails, 4 ft. 6 in. from center to center on the front face, and on the back face 4 ft. 6 in. from center to center for the lower 35 ft. of their length, and 9 ft. 0 in. from center to center for the upper 26 ft.

The horizontal reinforcement consists of five rings of 12-lb. rail immediately below the crest of the dam, and, from there down, $\frac{5}{8}$-in. round and square rods, with spacing varying from 15 to 18 in. from center to center.

The lower 30 ft. of the dam is not reinforced. The length of the dam, measured on the arc, is comparatively small in this zone, and the possible temperature range is materially less than in the higher portions. Practically, the front face of the dam for this depth will be constantly immersed, which tends to uniform temperature. On the back face the lower part is less exposed to the sun than the upper.

TABLE 6.—CONCRETE QUANTITIES.

		Cubic yards.
Main dam and outlet tower		4 093
Spillway weir	475 cu. yd.	
Spillway floor and retaining wall	308 " "	783
Water-cushion dam		82
Miscellaneous		30
Total		4 988

PIPE LINES.

Water from the reservoir is conducted to the mill tanks at Nacozari through a pipe line, 2.91 miles long, composed of 1.13 miles of 8-in. and 1.78 miles of 10-in. steel riveted pipe, both of No. 14 gauge, furnished by the Lacy Manufacturing Company, of Los Angeles. Only on the lower 3 600 ft. of the line could wagons be used for the distribution of the pipe; for the remaining distance the pipe had to be packed by animals and men. Under these conditions, the savings to be made by using light riveted pipe outweighed considerations of permanency.

For wagon transportation, the pipe was made up in 20-ft. sections, and for pack animals in 10-ft. sections. Where buried in the ground, the usual California drive joints were used, and where exposed, flanged joints. The flanges are of pressed steel, riveted to the pipe. The flange dimensions were: for 10-in. pipe, 14¼ in. in diameter, 12⅜-in. bolt circle, 12 ½-in. bolts; and, for 8-in. pipe, 12¼ in. in diameter, 10⅜-in. bolt circle, 10 ½-in. bolts. When pipe had to be cut, or articulation was needed, bolted joints of the loose-ring type, with triangular rubber gaskets, were used. For water-tightness, the drive joints are dependent on the asphalt coating which the pipes receive in the final process of manufacture. In the field, after entering the end of the driven pipe, kerosene is poured around the outside of the joint and fired, and if necessary refired, until the asphalt coating softens and runs; then the

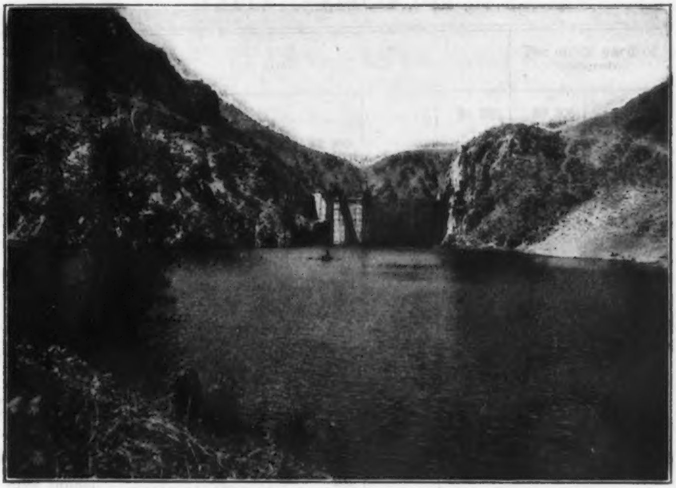


FIG. 16.—FRONT OF DAM, WITH 53 FEET OF WATER.

FIG. 17.—MAIN DAM, CUSHION DAM, AND SPILLWAY.

TABLE 7.—Cost of Dam.

					Per cubic yard of concrete.	
Roads and trails, labor				$1 528	$0.306	
Plant:						
Buildings, material		$2 985				
" labor		803	$3 788			
Sand bins, material		$970				
" labor		416	1 386			
Cableways, material		$1 574				
" labor		734	2 308			
Machinery and supplies, material		$4 363				
Installing machinery, labor		3 982				
Transporting machinery		1 190	9 535			
Water tanks			158	17 175	3.443	
Miscellaneous, unsegregated, material		$2 395				
" " labor		7 869		10 264	2.058	
Gates and control fittings, tower		$800				
Installing piping, tower, labor		880		1 680	0.337	
				$30 647		$6.144
Excavations:						
Outlet tunnel		$365				
Inlet tunnel		231				
Intake shaft		275	$871			
Spillway			2 264			
Foundation trenches, etc.			2 173	5 308		1.064
				$35 955		$7.208
Concrete:						
Cement at Nacozari		$26 840				
Transportation of cement		2 847	$29 687		$5.952	
Sand, screening and handling		$8 007				
Transportation of sand		3 430	11 437		2.293	
Coarse aggregate, crushed rock from spillway excavation. Cost of crushing not segregated.						
Forms:						
Lumber		$4 712				
Steel		2 600				
Labor		6 151				
Transportation		1 189	14 652		2.937	
Mixing and placing concrete, and crushing rock, labor		$6 541				
Fuel for above, coal	$1 318					
Transportation of coal	267					
Wood	2 048	3 633	10 174		2.040	
Blacksmithing, labor		$1 248				
" material		408	1 656		0.332	
Reinforcing material			1 454	69 060	0.291	$13.845
				$105 015		$21.053
Superintendence				2 790		0.559
Engineering, field			$2 815		$0.564	
" plans and supervision			3 670	6 485	0.736	1.300
Total				$114 290		$22.912

pipe is driven home. In straightaway work, a gang of experienced men can lay about 2 500 ft. of 8-in. or 2 200 ft. of 10-in. pipe in a day. When properly driven, and the pipe ends have not been deformed by transportation and handling, the percentage of leaky joints on testing out is very small. Leaky joints are remedied by galvanized-iron sleeves of sufficiently greater diameter than the pipe to leave an annular space of about $\frac{3}{4}$ in. into which neat cement or cement mixed with fine sand is rammed. When, from necessity, sleeves have to be put on a pipe under pressure, a closeable vent should be provided at the midlength of the sleeve to discharge all leakage water until the cement sets. Seam leaks of minor nature are curable by floating fine sawdust through the pipe. With large seam leaks, it is preferable to put in a new section of pipe.

TABLE 8.—COST OF PIPE LINE.

Huacal pipe line	5 974 ft. 8-in. pipe.
	9 415 " 10- "
Total length	15 389 ft.

		Total cost.	Average cost per linear foot.
Material:			
Pipe and fittings	$12 456		
Trestles	1 857		
Grading tools and supplies	561		
Transportation of material	790	$15 664	$1.02
Labor:			
Grading	$4 546		
Framing and erecting trestles	1 051		
Laying and covering pipe	2 056	7 653	0.49
		$23 317	$1.51
Superintendence		310	0.08
Engineering, field	$950		
" designing and supervision	816	1 766	0.11
Total		$25 393	$1.65

Break-pressure tanks of concrete, with "Davis" float-valve control, are introduced at two convenient points on the line, which limit the maximum static pressure to 179 ft. for the 8-in. and 283 ft. for the 10-in. pipe. At the summits, air relief and prevention of vacuum are afforded by 2-in. pipes carried up the mountain side to an elevation above the static head. The open ends of these pipes are protected by wire gauze to prevent the intrusion of foreign substances and living

things. Where the situation did not permit of open pipes, "Simplex" combined air relief and vacuum valves were used. The breakage of the pipe or the sudden opening of a blow-off valve at a low point in the line, thereby causing a partial vacuum, are to be particularly guarded against in light pipes. A vacuum which might be without ill effect in a heavy metal pipe line is absolutely fatal to thin riveted pipes, which have but little strength to resist pressure from without.

TEST OF PIPE CAPACITY.

Color tests of the velocity of the water traversing the pipe line were made by using permanganate of potash. Through 4 354 ft. of the 8-in. line, with a fall of 170 ft., the velocity was found to be 7.223 ft. per sec., from which the discharge was computed as 2.521 sec-ft.; the corresponding value of n in Kutter's formula is 0.0114. The velocity through 9 415 ft. of the 10-in. pipe, with a fall of 110 ft., was found to be 4.689 ft. per sec., and the computed discharge 2.555 sec-ft.; and the value of n is 0.0116. A volumetric test, made at the mill tank, gave the discharge as 2.529 sec-ft. In view of the fact that the pipes and tank were not calibrated, the three determinations of 2.521, 2.555, and 2.529 cu. ft. per sec. may be said to be in perfect agreement, the difference between the highest and lowest determination being but 1.33 per cent.

The writer takes this occasion to express his thanks to Mr. J. S. Williams, Jr., General Manager, and to other officers of the Moctezuma Copper Company, for their aid and co-operation in the work, and to Mr. C. A. Dodge for his effective supervision of the work of construction.

DISCUSSION

Mr. Sherman.

CHARLES W. SHERMAN,* M. AM. SOC. C. E. (by letter).—The writer has long felt that the arched dam is worthy of much greater attention than it has usually received. No one has ever shown how such a dam could fail, if the abutments were sound and the crushing strength of the arch and abutments was not exceeded; and, as a matter of fact, as noted by Mr. Hawgood, there does not appear to be any record of failure of a dam constructed on this principle.

The total number of dams in which dependence is placed on arch action for stability is so small that it seems desirable that a complete list be prepared. The author has overlooked several American arched dams, two of which rank next to the Bear Valley and Upper Otay in boldness, as measured by intensity of stress in the arch. The dams referred to are those at Lewiston, Idaho, and Winchester, Ky., built by William Wheeler, M. Am. Soc. C. E.

Table 9 contains a complete list of arched dams, as far as the writer has been able to ascertain, and includes data on ten dams not given in Table 1.

It is particularly interesting to note the test of a brick arch, 4 in. thick, with a radius of 58 ft. 1 in., comprising a facing wall of the Ithaca Dam, which occurred during the construction of the dam, as described by Gardner S. Williams, M. Am. Soc. C. E.† The arch was 52 in. high, with a clear span of 90 ft. and with a head of water of 0.8 ft. over the top. According to the ordinary method of arch analysis, the stress in the arch at the base would have been 385 lb. per sq. in.

Las Vegas Dam.—In 1910-11, the Agua Pura Company, of Las Vegas, N. Mex., constructed‡ a concrete arched dam, 50 ft. high, to form a storage reservoir. It was designed to be increased in height to 95 ft., subsequently, and it was computed that, even if there should be no bond between the old and new work, and the full water pressure should be transmitted through the resulting joint, the maximum stress in the new work would not exceed 520 lb. per sq. in.

The design of this dam has been described briefly by the writer,§ and its construction by William T. Barnes, M. Am. Soc. C. E.||

As the drainage area above the dam was very small, and a diversion ditch has been constructed by which the run-off in times of rain will be conducted out of the basin, it was thought possible to build the dam without a spillway, and to provide for the storage of water up to the level of the crest of the dam. The water is to be supplied to the

* Boston, Mass.

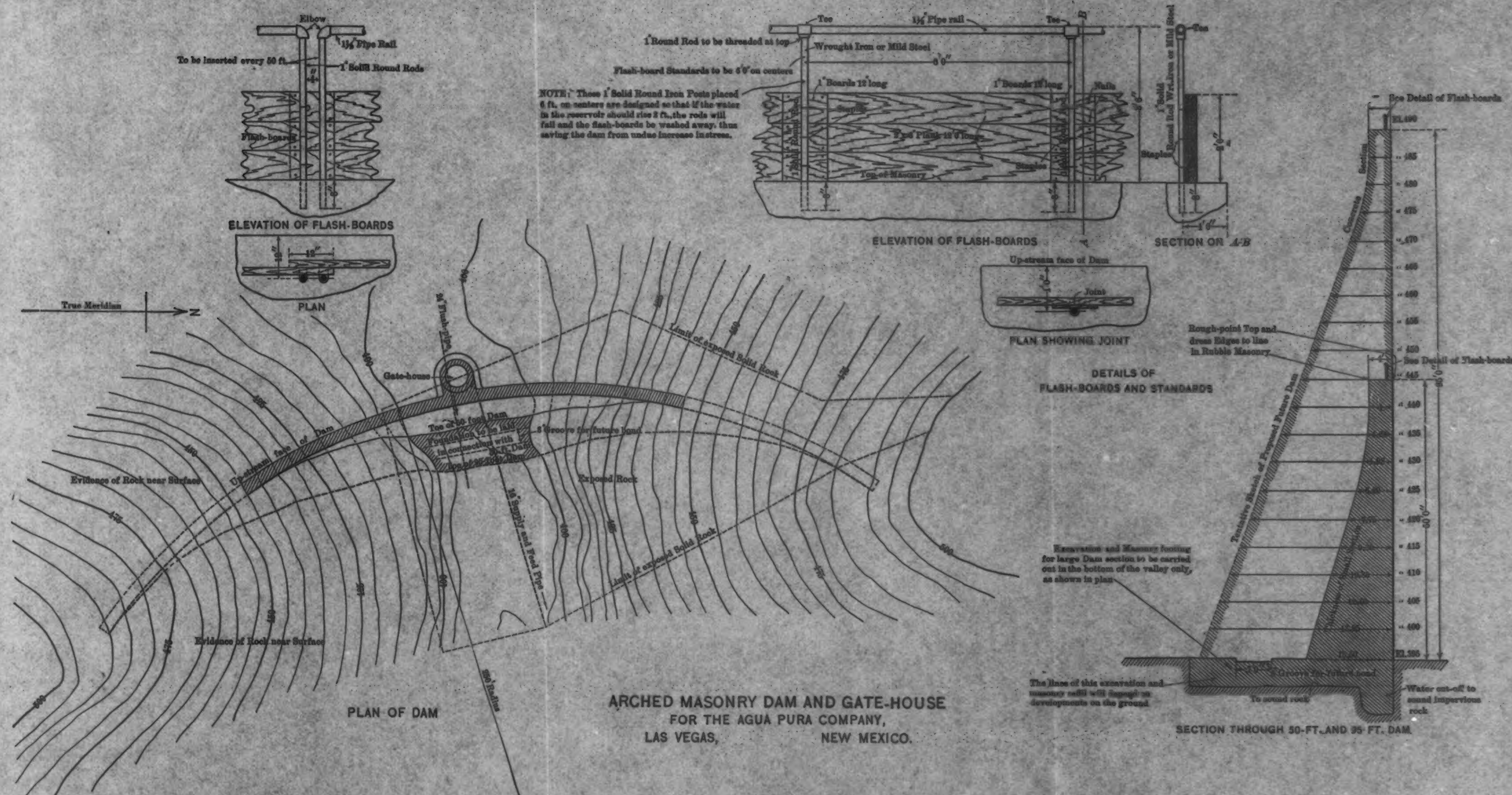
† *Transactions*, Am. Soc. C. E., Vol. LIII, pp. 199-200.

‡ From designs by Metcalf and Eddy.

§ *Engineering News*, Oct. 27th, 1910.

|| *Journal*, New England Water Works Assoc., Vol. 26, p. 356.

PLATE XII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. [illegible].
SHERMAN ON
CONCRETE ARCH DAM.



ARCHED MASONRY DAM AND GATE-HOUSE
FOR THE AGUA PURA COMPANY,
LAS VEGAS, NEW MEXICO.

Mr. Sherman.

TABLE 9.—PRINCIPAL DETAILS OF CURVED MASONRY DAMS.

Reference.	Place.	Maximum height, in feet.	Thickness at base, in feet.	Thickness at top, in feet.	Maximum stress in arch, pounds per square inch.	Radius of up-stream face, in feet.	Length at top, in feet.	Character of rock.	Date.
(1)	Katoomba, N. S. W.	25	20.29	3.0	233	220	320	Sandstone	1905
(6)	Cascadilla Cr., Ithaca, N. Y.	25	40.0	2.5	189	70	96	Shale	1904
(1)	Picton, N. S. W.	28	13.62	7.0	186	120	112	Sandstone	1897
(3)	Winchester, Ky.	31	8.58*	4.83	498	318.4	407	....	....
(1)	Qn. Charlotte Vale, N. S. W.	32	8.65	3.0	155	90	113	Quartzite	1898
(1)	Parkes, N. S. W.	33.5	13.5	3.0	373	300	540	Granite	1897
(1)	Lithgow No. 1, N. S. W.	35	10.88	3.5	155	100	178	Sandstone	1896
	Barren Jack, N. S. W.	38	5.00	....	264	80	....	....	1907
(12)	Meer Allum, India	39.0	8.5	....	164	82	178	....	1800±
(1)	Wollongong, N. S. W.	42	11.62	3.5	311	200	585	Basalt	....
(1)	Cootamundra, N. S. W.	46	13.0	3.0	389	250	640	Granite	1898
(1)	Wellington, N. S. W.	48	10.0	3.0	311	150	350	Conglomerate	1899
(2)	Bear Valley, Cal.	48 (total 64)	8.4 at 48 ft.	3.17	825	335	300	Granite	....
(1)	Mudgee, N. S. W.	50	18.0	3.0	311	253	498	Altered slate	1899
(11)	Las Vegas, N. Mex. (built)	50	15.50	4.0	350	250	210	....	1910
	" " " (proposed)	95	43.30	4.0	300	250	390	Sandstone	....
(1)	Parramatta, N. S. W.	52	15.0	4.8	223	160	225	Sandstone	....
(3)	Lewiston, Idaho	55.5	14.5	5.33	475	286.5	288	....	....
(8)	Crowley Cr., Malheur Co., Ore.	60	5.2	3.0	350	70	170	....	1908
(13)	" (projected, 1914)	90	9.2	3.2	305	72	223	Lava	....
(1)	Tamworth, N. S. W.	61	21.5	3.0	311	250	440	Granite	1898
(9)	Goodwin Dam, Stanislaus River, Cal.	61 (total 70)	12.0	8.0	297	135	233	....	1913
(1)	Medlow, N. S. W.	65	8.96	3.5	186	60	124	Sandstone	1906
(7)	Upper Otay, Cal.	75	14	4	604	359	350	....	1900
(1)	Lithgow No. 2, N. S. W.	87	24.0	3.0	155	100	221	Sandstone	1906
(10)	Huacal, Sonora, Mex.	88.5	12.83	3.5(?)	226	76	140	Andesite	1912
(2)	Sweetwater, Cal.	90	46	12	188	222	380	....	....
(4)	Six-Mile Creek, Ithaca, N. Y.	90†	7.75	....	283	67.85	....	Shale	1903
(2)	Barossa, S. Australia	95	34	4.5	242	200	....	....	1903
(2)	Zola, France	119.7	41.82	19.02	196	158	205	....	....
(4)	Lake Cheesman Dam, Colo.	210 to H. W.	176 72.66 at El. 100	18	....	399.89 at top	....	Granite	1900
(5)	Pathfinder, Wyo.	210	94	10	181	150 at center 186.5 at base	....	....	....

(1) *Engineering News*, May 19th, 1910.
(2) Wegmann's "Dams," 5th Edition.
(3) Data obtained from William Wheeler, M. Am. Soc. C. E., Engineer.
(4) *Transactions*, Am. Soc. C. E., Vol. LIII.
(5) *Journal*, N. E. W. W. Assoc., June, 1906.
(6) *Engineering News*, November 10th, 1910.
(7) Schuyler's "Reservoirs," 2d Edition.
(8) *Engineering News*, August 24th, 1911, p. 220.
(9) *Engineering News*, October 16th, 1913.
(10) *Proceedings*, Am. Soc. C. E., April, 1914.
(11) *Engineering News*, October 27th, 1910; and *Journal*, N. E. W. W. Assoc., December, 1912.
(12) *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 214.
(13) *Engineering Record*, June 30th, 1914, p. 693.

* 15 ft. above base. † As designed; actually built only 30 ft. high.

Mr. Sherman. reservoir partly through a canal and partly through a pipe which also serves as an outlet. It was believed, therefore, that the likelihood of the water rising higher than was desired would be very small indeed, and that, in case of a cloudburst or other emergency causing the water to overtop the dam, no serious damage was to be apprehended. Fig. 18 is an isometric view of the forms as used in building the dam.

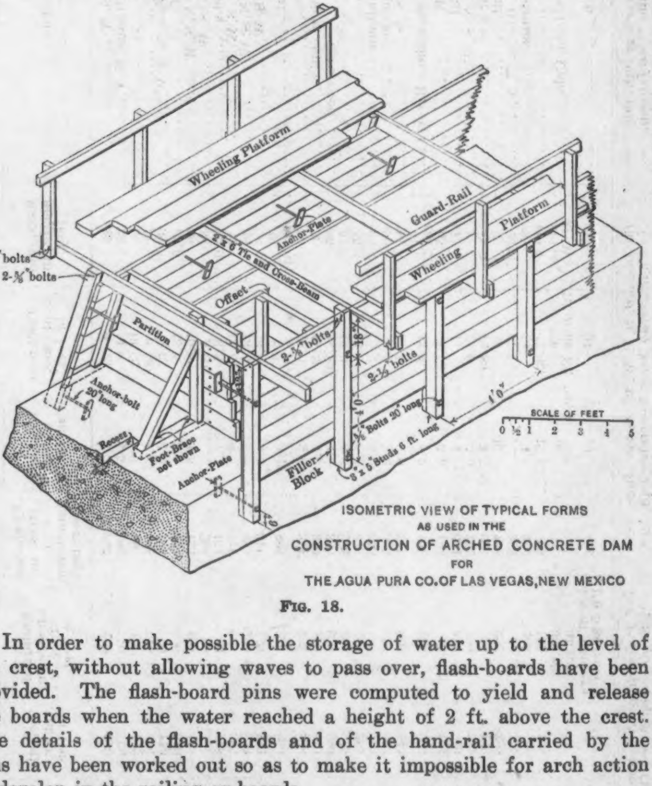


FIG. 18.

In order to make possible the storage of water up to the level of the crest, without allowing waves to pass over, flash-boards have been provided. The flash-board pins were computed to yield and release the boards when the water reached a height of 2 ft. above the crest. The details of the flash-boards and of the hand-rail carried by the pins have been worked out so as to make it impossible for arch action to develop in the railing or boards.

The arrangement of the gate-house, Fig. 21, is believed to be novel, particularly in its compactness and the method adopted for screening the outlets at different depths by the use of a single section of screen.

Plate XII shows the plan and section of the dam and gate-house, and the appearance of the structure when substantially finished is shown by Figs. 19 and 20.

DAM AT LAS VEGAS, N. MEX., FOR AGUA PURA COMPANY.

Fig. 19.—Top of Dam Nearly Completed.

Fig. 20.—Up-stream Face of Completed Dam.

Mr. Sherman.

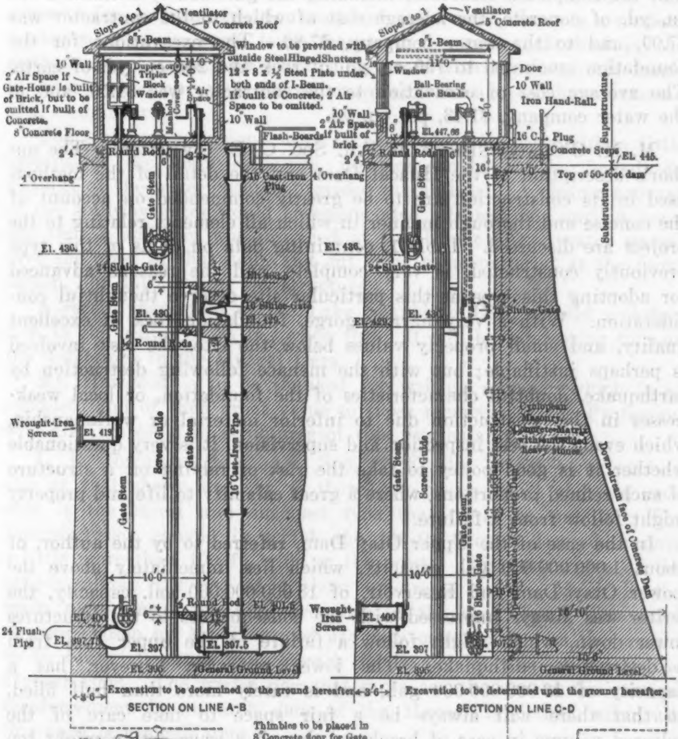


FIG. 21.

Mr. Sherman.

The methods and costs of construction were stated by Mr. Barnes in the paper before referred to. It may be of interest to note here that the completed structure, including the gate-house, contains 2 703 cu. yd. of concrete, the average cost of which to the contractor was $7.09, and to the water company, $7.86. The excavations for the foundation amounted to 790 cu. yd. of rock and 245 cu. yd. of earth. The average cost of excavation to the contractor was $1.50, and to the water company, $1.43, per cu. yd.

Mr. O'Shaughnessy.

M. M. O'Shaughnessy,* M. Am. Soc. C. E. (by letter).—The author's description of the Huacal Dam and the detail of the methods used in its construction are to be greatly commended on account of the concise and thorough manner in which all elements relating to the project are discussed. Table 1, containing data on dams of this type previously constructed, is very complete, and the reasons advanced for adopting this type at this particular site deserve thoughtful consideration. With a very narrow gorge, foundation rock of excellent quality, and small property values below the site, the risk involved is perhaps justifiable; but with the menace following destruction by earthquake, doubtful characteristics of the foundation, or local weaknesses in the construction due to inferior material or workmanship, which evade the best inspection and supervision, it is very questionable whether it is good policy to take the risk of relying on a structure of such refined proportions, where a great calamity to life and property might follow from a failure.

In the case of the Upper Otay Dam, referred to by the author, of about 1 000 000 000 gal. capacity, which lies immediately above the Lower Otay Dam and Reservoir, of 13 000 000 000 gal. capacity, the writer was always impressed with the consequences to the structures lower down, which might follow a failure of the upper dam from a destructive earthquake. The lower reservoir, however, has a capacity of 13 000 000 000 gal., and is rarely more than half filled, so that there will always be a fair space to take care of the released volume in case of breakage, though a wave action might top the lower dam or crowd the spillway to its capacity.

In dam construction, engineers on the Pacific Coast are often compelled to adopt economies of design which older and richer communities can afford to reject; in other words, they are very often compelled to adopt boldness of design, which, under other conditions, they might not accept.

The cost of the Huacal Dam was $114 000, excluding water rights or reservoir site, and as the capacity is 2 700 acre-ft., the cost was $42 per acre-ft., which is very reasonable for a structure of this type, with a small reservoir basin and a limited water-shed. The concrete

* San Francisco, Cal.

Mr. O'Shaughnessy.

work in the structure must have been well done, as the leakage is only 6 000 gal. per 24 hours; and the precautions adopted by Mr. Hawgood to obtain a superior concrete mix are to be commended. In concrete for dam construction and hydraulic work, there is a modern tendency, even in some large works, to be rather slipshod in the methods, precautions, and proportions used, with the inevitable result of after consequences to the structure and damage to the reputation of the engineers.

The spillway provisions seem to be ample, but, even with a little topping of the main structure, injuries should not be feared, on account of the water-cushion provided at the base. A clearer account of the spillway gates would be appreciated. It was not until 5 years ago that a spillway was provided at the Upper Otay; water passed over the crest of that dam, and cut away some material at the foundation, but did not hurt the structure.

A recent inspection of the Lake Spaulding Dam showed a deflection in the center of 3½ in., when full. It will be interesting to note the deflections in the Huacal Dam.

Mr. Hawgood is to be thanked for writing a paper which is so complete and of such great value to those members of the Profession who are interested in structures of this particular type.

Mr. Jorgensen.

L. R. Jorgensen,* M. Am. Soc. C. E. (by letter).—For such a site as that shown on Fig. 5, there can be no doubt that an arch dam is the safest and cheapest type that could be put in. The (approx.) box canyon in which the 100-ft. dam is located is only a little more than 100 ft. wide at the crest. This narrowness of the canyon allows the use of a short up-stream radius, which is the main requirement for high economy in the arch dam. The axial stresses chosen by Mr. Hawgood are very conservative, as they ought to be for a structure having such fairly slender proportions as the Huacal Dam. The writer does not quite see the use of the vertical reinforcement in an arch dam of the Huacal type, but, after all, this steel is a small item.

It would be better if engineers in general referred to dam faces as the up- and down-stream faces instead of front and rear faces, as this latter definition is not as clear as the former and is often used inconsistently.

Mr. Hawgood has built a very good substantial structure and need not make any excuses for putting in an arch dam. An engineer who would recommend any other type for the site in question would have a hard time finding enough excuses to justify such a decision.

With the exception of the formula giving the down-stream deflection of the crown of the dam under load, all other formulas used in

* San Francisco, Cal.

Mr. Jorgensen.

arch dam design, though they are not absolutely accurate, are as accurate as those ordinarily used in the field of practical mechanics.

In dam design the load is definitely known; it is not, however, definitely known where this load applies itself, unless the up-stream face is provided with a fairly water-tight skin, such as a plaster put on with a concrete gun. This is one reason why dams having a slender section ought not to be stressed (unit stress) as high as arch dams having a heavier section.

In the formula for the deflection of the crown, the modulus of elasticity of the concrete appears. Some value for this has to be substituted in the formula, and that is where the biggest assumption in dam design is apt to be made.

To know, beforehand, what the deflection of the dam, when loaded, will be, is of small real importance, and if it turns out to be different from the calculated deflection, it is thereby indicated that the modulus of elasticity of the mass of concrete is different from the assumed one. The modulus of elasticity appears in other formulas, but in such a way that it cancels, and, therefore, no assumptions as to its actual value need be made.

According to the writer's view, an arch dam can be calculated as accurately as most structures, and when it is considered how far from its crushing strength the material is ordinarily working, such a structure should be readily accepted by all engineers as the safest, as well as the cheapest, that can be built in such a place as that described by Mr. Hawgood.

The automatic overflow gates provided for the Huacal Dam are very simple and evidently effective, and undoubtedly will give satisfaction.

Mr. Mensch.

L. J. Mensch,* M. Am. Soc. C. E. (by letter).—The Huacal Dam and the many others mentioned in this paper should convince the most skeptical engineer that arched dams with cross-sections much lighter than those of gravity dams, have a fair factor of safety. None of these structures ever failed or showed larger or more numerous cracks than those found in gravity dams. Arched dams, being considerably cheaper than gravity dams, would be often used in place of earthen dams but for the fact that engineers have no confidence in the rather primitive, although evidently adequate, method by which the pioneer builders determined the cross-section of these structures. Engineers are afraid that some hidden defects exist in these dams, for which they cannot satisfactorily account by theory and for which they do not care to become responsible.

The objections to the assumption that arched dams act like cylinders subjected to external pressure are the following:

* Chicago, Ill.

First.—These dams are arches fixed at the abutments, and like other arches are subjected to stresses due to the shortening of the arch, change of temperature, and shrinkage and swelling of the concrete. When the reservoir is filled, the swelling of the concrete and the fall of temperature from the original temperature of setting counteract each other to a great extent, and only in case of the reservoir being empty there may exist the most unfavorable condition of a fall of temperature and shrinkage of concrete acting together with the tendency to produce cracks. Mr. Mensch.

Second.—It is evident that these dams act partly as gravity structures, and the unknown extent to which they act as such is the main objection of thoughtful engineers to this type. In the discussion* on the paper by G. N. Houston, M. Am. Soc. C. E., on the Halligan Dam, the writer has shown that the greatest portion of the water pressure is resisted by gravity action. He is now obliged to confess that he overlooked an important point. He assumed that the dam acted as a gravity structure entirely within the limit of ordinary beam action, following Hooke's law. This is not the case in masonry dams after the resultant of forces strikes the base outside the middle third, or in a reinforced structure after the stresses in the steel bars reach the elastic limit. In both cases after cantilever action is nearly exhausted we have the condition of the dam being rather hinged at the bottom than immovably fixed, as formerly assumed by the writer, and if this condition is assumed, theory gives results which agree exceedingly well with practice.

It is an acknowledged and very plausible law of Nature that, among all possible deformations which a structure subjected to exterior forces can perform, deformation will take place, which will make the work of the internal forces a minimum.

It can be proved that the work of compressing the structure as an arch (as long as the water level remains not higher than about 30 to 40% of the total depth of the water above the base of the dam) is much greater than that necessary to deflect the structure as a cantilever, hence gravity action must be the preponderant influence by which the stresses at the bottom of the dam are governed when the reservoir is about half full.

If, instead of an arch, we assume a complete cylinder like a water tank, it follows that the walls of the tank cannot move, even if set on a frictionless base, before the cantilever action is exhausted. The futility of all devices which were ever patented to disconnect walls of tanks from the base is thereby shown.

It can be shown that in all arched dams, with the possible exception of the Zola Dam, the lower portion of the structure, of a

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 135.

Mr. Mensch.

height of about 5% of the total depth of water, is strained far beyond anything possible in a gravity structure, yet no harm can come to the dam, because when gravity action ceases, arch action takes its place.

The author thinks that both the rock at the base and at the abutments of the dam moves in accordance with the dam, and hints that this may explain his success. Any rock which would allow such movements would be entirely unsuitable for an important arch structure, and to assume that a very much larger mass of rock than the mass of the dam allows anything but infinitesimal deformations, is out of the question.

The problem of the arched dam is statically indeterminate, and the stresses can only be found by taking the deformation of the arch into consideration.

Notation.—

Let w = the water pressure, in pounds per square foot;
H = the height of the dam, in feet;
h = the depth of water above the section in consideration, in feet;
r = the outside radius, in feet;
Δr = the change of radius due to forces acting on the dam;
d = the thickness of the arch, in feet, at any section;
d' = the thickness of the arch, in feet, at the base;
I = the moment of inertia of any section, in feet;
I' = the moment of inertia of the section at the base, in feet;
L = the length of the arch, in feet;
f = the versed sine of the arch;
E = the modulus of elasticity of the masonry per square foot, in pounds, $= 288 \times 10^6$;
T = the thrust of the arch, in pounds per linear foot in depth;
G = the weight of a slice of the dam 1 ft. long;
n = the distance of the down-stream face of the arch to the intersection of the resultant of forces with the base, in feet;
A' = the work of the interior forces, in foot-pounds per linear foot of the height of the dam, when the structure deforms as an arch only;
A'' = the work of the interior forces when the arch deforms as a girder only;
A = the work of a slice of the arch 1 ft. long and H feet high when the dam deforms as a cantilever only;
P = the force, in pounds per linear foot of the height of the dam, due to arch action only;
M = any moment, in foot-pounds;
S = any stress, in pounds per square foot;

t = a change in temperature, here assumed to be 50° Fahr.; Mr. Mensch.

c = the coefficient of expansion, 0.0000055;

$c \, . \, t$. = the total relative expansion or contraction, equal to $\frac{1}{3\,640}$;

x and y = co-ordinates from a point of origin identical with the left abutment;

dx, dy, ds = increments of x, y, and the length of the arch;

$\Delta x', \Delta y', \Delta \rho'$ = the changes of the ordinates of any special point, x', y', and of the angle, ρ', which the tangent at x', y', forms with a line connecting the two abutments.

For curved beams, where the radius is from five to ten times larger than the depth of the beam, the textbooks give the following equations for the change of the co-ordinates of any point, x', y', of the arch when P is a force normal to any section, x, y, and M is the bending moment about x, y,

$$\Delta x' = \int_0^{x'} \frac{P\,dx}{E\,F} + \int_0^{x'} \frac{M\,ds}{E\,I} - \int_0^{x'} \frac{M\,y\,ds}{E\,I} \quad \ldots\ldots\ldots(1)$$

$$\Delta y' = \int_0^{x'} \frac{P\,dy}{E\,F} - x' \int_0^{x'} \frac{M\,ds}{E\,I} + \int_0^{x'} \frac{M\,x\,ds}{E\,I} \quad \ldots\ldots\ldots(2)$$

$$\Delta \rho' = \int_0^{x'} \frac{M\,ds}{E\,I} \quad \ldots\ldots\ldots(3)$$

Where the radius of the curved beam is about twice the depth of the beam, the foregoing equations give values about 10% larger than the exact formulas.

The writer also makes the customary assumption that the abutments are fixed and do not admit of any displacements or change of angle of the arch at the abutment.

Most arched dams subtend an angle of about 90°; the writer has shown* that an angle of 120° is the most favorable one. For simplification, all the following investigations will relate to these two angles only.

Influence of a Change of Temperature on the Arch When the Reservoir is Empty.—For a fall of temperature, for example, the arch has a tendency to shorten its length and span, but this is prevented by a pull or negative thrust from the abutments and by the moments, Ma, exerted by the abutments on the arch in order to keep the angles at these places from changing. No forces normal to the chord can be exerted by the abutments, because there are no exterior forces acting on the dam which can hold them in equilibrium. From Fig. 22 it follows that the moment about any point, x, y, is

$$M = T\,y - Ma \quad \ldots\ldots\ldots(4)$$

* "Reinforced Concrete Pocket Book."

Mr. Mensch. As the assumption is made that the abutments are fixed and do not allow any change of angle between them, it follows from Equation (3) that

$$\int_0^L \frac{M\,ds}{E\,I} = 0.$$

On account of the symmetry of the arch and the loading,

$$\int_0^L \frac{P\,dy}{E\,F} = 0,$$

hence Equation (2), taken between the limits, 0 and L, reduces to

$$\int_0^L \frac{M\,x\,ds}{E\,I} = 0,$$

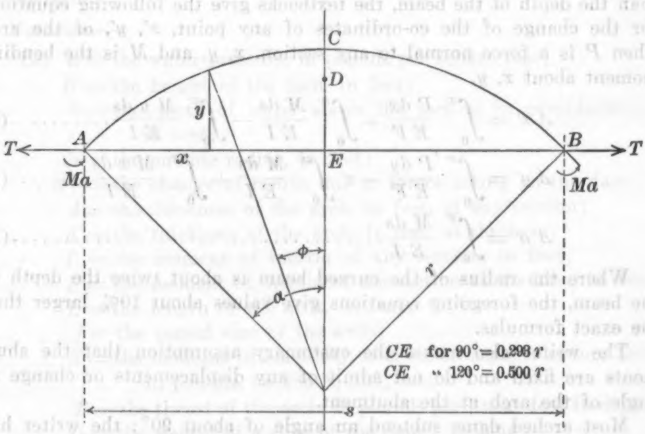


FIG. 22.

or, on account of Equation (4),

$$\int_0^L \frac{T\,y\,x\,ds}{E\,I} - \int_0^L \frac{Ma\,x\,ds}{E\,I} = 0,$$

which condition is fulfilled when

$$Ma = T\,\overline{D\,E} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

when $D\,E$ is the distance of the center of gravity of the arch from $A\,B$. Equation (1) now becomes

$$\Delta x' = \Delta s = \int_0^L \frac{P\,dx}{E\,I} - \int_0^L \frac{M\,y\,ds}{E\,I}$$

$$= \int_0^L \frac{P\,dx}{E\,I} - \int_0^L \frac{T\,y^2\,ds}{E\,I} + \int_0^L \frac{D\,E\,T\,y\,ds}{E\,I}.$$

Mr. Mensch.

$E, F, I, \overline{DE}$, and T are constants; hence

$$\Delta s = \frac{1}{E F}\int_0^L P\,dx - \frac{T}{E I}\int_0^L y^2 ds + \frac{\overline{DE}\,.\,T}{E I}\int_0^L y\,ds \ldots (6)$$

The integration is facilitated by using polar co-ordinates, as shown in Fig. 22.

$$x = r\,(\sin.\ \alpha - \sin.\ \rho)$$
$$y = r\,(\cos.\ \rho - \cos.\ \alpha)$$
$$dx = -r \cos.\ \rho\ d\rho$$
$$ds = -r\,d\rho$$
$$P = T \cos.\ \rho$$
$$s = 2\,r \sin.\ \alpha$$

$$\Delta s = c\,.\,t\,.\,s\,. = \frac{1}{3\,640}\,s.$$

It is easily found that $\overline{DE} = 0.193\,r$ for 90°;

and $\overline{DE} = 0.323\,r$ for 120°.

Hence, for an arch subtending 90°, Equation (6) becomes

$$-c\,.\,t\,.\,s\,. = -\frac{2\,T\,r}{E F}\int_{45}^{0} \cos.^2 \rho\,d\rho + \frac{2\,T\,r^3}{E I}\,(\cos.\ \rho - \cos.\ \alpha)^2\,d\rho$$
$$-\frac{2\,T\,0.193\,r^3}{E I}\,(\cos.\ \rho - \cos.\ \alpha)\,d\rho.$$

The integrals may be found in any handbook on calculus, and

$$-\frac{1}{3\,640}\,2\,r \sin.\ \alpha = -\frac{2\,T\,r}{E F}\left[\frac{1}{4}\sin.\ 2\,\rho + \frac{\rho}{2}\right]_{45}^{0}$$
$$+\frac{2\,T\,r^3}{E I}\left[\frac{1}{4}\sin.\ 2\,\rho + \frac{\rho}{2} - 2\cos.\ \alpha \sin.\ \rho + \rho \cos.^2 \alpha\right]_{45}^{0}$$
$$-\frac{2\,T\,r^3}{E I}\,0.193\left[\sin.\ \rho - \rho \cos.\ \alpha\right]_{45}^{0}$$

$$-\frac{1}{3\,640}\,2\,r\,(0.707) = \frac{T\,r}{E F}\,1.285 + \frac{T\,r^3}{E I}\,(0.07) - \frac{T\,r^3}{E I}\,(0.0586) \ldots (7)$$

$$F = d,\ I = \frac{d^3}{12},\ \text{and}\ T = \frac{816\,000}{1 + 9.4\,\frac{d^2}{r^2}}\,\frac{d^3}{r^2}.$$

The moment at the crown is

$$0.100\,r\,T = 81\,600\,\frac{d^3}{r}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}} \ldots\ldots\ldots\ldots (8)$$

The moment at the abutment is

$$0.193\,r\,T = 157\,500\,\frac{d^3}{r}\,\frac{1}{1 + 9.4\,\frac{d^2}{r^2}} \ldots (9)$$

Mr. Mensch.

The moment of resistance of a ring of the arch, 1 ft. high and d feet thick, equals $\frac{d^2}{6}$, and the stresses in the extreme fibers of the crown section equal

$$489\ 600 \frac{d}{r} \frac{1}{1 + 9.4 \frac{d^2}{r^2}} \qquad \text{(10)}$$

And those at the abutments equal

$$945\ 000 \frac{d}{r} \frac{1}{1 + 9.4 \frac{d^2}{r^2}} \qquad \text{(11)}$$

For an arch subtending an angle of 120°, we obtain, similarly,

$$T = \frac{228\ 500}{1 + 2.47 \frac{d^2}{r^2}} \frac{d^3}{r^2} \qquad \text{(12)}$$

And the stresses in the extreme fibers at the crown equal

$$\frac{242\ 300}{1 + 2.47 \frac{d^2}{r^2}} \frac{d}{r} \qquad \text{(13)}$$

The stresses in the extreme fibers at the abutments equal

$$\frac{442\ 800}{1 + 2.47 \frac{d^2}{r^2}} \frac{d}{r} \qquad \text{(14)}$$

These stresses are often very high, especially in thick dams, and are increased considerably by the shrinkage of the concrete, which may amount to $\frac{1}{2\ 500}$ of the length in a year. The assumed shortening of the arch under a fall of 50° Fahr. was $\frac{1}{3\ 640}$ of its length, hence the shrinkage stresses amount to nearly $1\frac{1}{2}$ times as much. There is actually no way of preventing cracks, and the best practice is to provide vertical joints along radial planes at distances of about 50 ft., which will have the effect of concentrating the cracks in the least dangerous places, and to make the cracks between the joints very fine.

The Dam as a Pure Arch Structure.—Assuming the arch to be a portion of a thin cylinder acted on by exterior water pressure, it is known that the diminution of the radius,

$$\Delta r = \frac{w\, h\, r}{d\, E} \frac{2 \pi r}{2 \pi} = \frac{w\, h\, r^2}{d\, E} \qquad \text{(15)}$$

The arch would take the position, $A'\ C'\ B'$, if not restrained. It can be imagined that the restraint will cause the arch first to take the position, $A''\ C''\ B''$, and then, through a negative thrust, T, and a moment, Ma, to take the position, $A\ F\ B$.

This behavior is commonly called the shortening of the arch, and represents the secondary action of the arch as a common girder, fixed at its ends, due to the deflection of the arch, and is practically identical with that caused by a fall of temperature; the unknown value, T, can be obtained by substituting in Equations (6) and (7), $2 \Delta r \sin. \alpha$ for Δs. Mr. Mensch.

Hence, $1.414 \Delta r = \frac{T r^3}{E I} 0.0114 \left(1 + 9.4 \frac{d^2}{r^2}\right)$ for a 90° arch.

Substituting for Δr the value in Equation (15), and for I, $\frac{d^3}{12}$,

$$T = 10.33 \frac{d^2 w h}{r} \frac{1}{1 + 9.4 \frac{d^2}{r^2}} \qquad \text{(16)}$$

The moment due to the shortening of the arch at the crown

$$= 0.100\, T r = 1.033 \frac{d^2 w h}{1 + 9.4 \frac{d^2}{r^2}}.$$

The moment due to the shortening of the arch at the abutments

$$= 0.193\, T r = 1.95 \frac{d^2 w h}{1 + 9.4 \frac{d^2}{r^2}}.$$

The extreme fiber stresses due to shortening at the crown are obtained by dividing the moment by $\frac{d^2}{6}$,

$$6.2\, w h \frac{1}{1 + 9.4 \frac{d^2}{r^2}}.$$

And the extreme fiber stresses due to shortening at the abutments,

$$11.7\, w h \frac{1}{1 + 9.4 \frac{d^2}{r^2}}.$$

The direct compressive stresses from the thrust due to water pressure are

$$w h \frac{r}{d}.$$

In most of the dams cited by the author, $\frac{r}{d}$ is about 20, hence the maximum compressive stresses due to arch action and the shortening of the arch are from 30 to 60% greater than those ordinarily assumed. In the Huacal Dam, $\frac{r}{d} = 6$, hence the maximum stresses are at least double those assumed.

Mr. Mensch.

For arches subtending an angle of 120°, the maximum fiber stresses from the shortening of the arch at the crown are

$$3.072\, w\, h \frac{1}{1 + 2.47 \frac{d^2}{r^2}}$$

And the maximum fiber stresses at the abutment

$$5.61\, w\, h \frac{1}{1 + 2.47 \frac{d^2}{r^2}}.$$

The distance, $C''\,F$, in Fig. 23, can be found from Equation (2) by omitting the second expression, which vanishes, as heretofore explained.

$$\Delta\, y = \int_0^{\frac{L}{2}} \frac{P\,dx}{E\,F} - \int_0^{\frac{L}{2}} \frac{M\,x\,ds}{E\,I}.$$

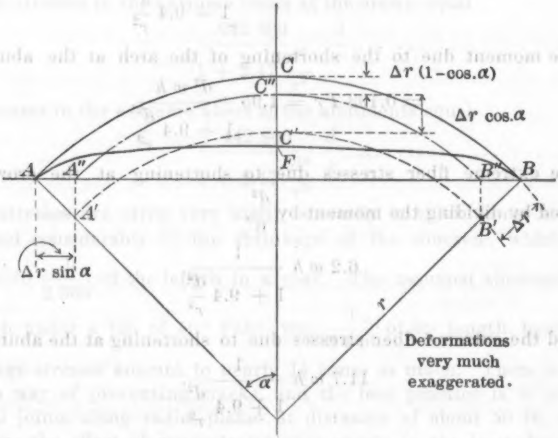


FIG. 23.

Introducing again polar co-ordinates, and $M = T\,.\,y - Ma$.

And, for 90°, $Ma = 0.193\ T\,.\,r$

$$\Delta y = C''\,F = -\int_{45}^{0} \frac{T\,r}{E\,F} \cos.^2 \rho\, d\,\rho$$

$$-\int_{45}^{0} \frac{T\,r^3}{E\,I} (\cos.\,\rho - 0.9)\,(\sin.\,\alpha - \sin.\,\rho)\,d\,\rho$$

$$C'\,F = \frac{T\,r}{E\,F}(0.6425) - \frac{T\,r^3}{E\,I}(0.014) = 0.014\,\frac{T\,r^3}{E\,I}\left[1 + \frac{I}{r^2 F}\,\frac{0.6425}{0.014}\right]$$

Mr. Mensch.

The value of T is given by Equation (16), and, neglecting the expression containing $\frac{I}{r^2 F}$,

$$C' F = 10.33 \frac{d^2 w h}{r} \frac{r^3}{E I} (0.014) = 1.735 \frac{w h r^2}{E d} = 1.735 \Delta r$$

and, from Fig. 23, it follows directly

$$C C'' = 0.293 \Delta r$$

or $C F$ equals the total deflection for 90°

$$= 2.028 \Delta r \qquad (17)$$

Similarly, the total deflection for 120°

$$= 1.66 \Delta r.$$

Work of Deformation in an Arch Structure.—The work performed by the interior forces of an arch may be divided into two parts, namely, that performed by the direct compressive stresses, and that by the moments due to the shortening of the arch. The textbooks give the following formulas for the work of the interior forces of a prism subjected to compressive or tensile forces and for the work performed by a girder subjected to bending moments.

$$A' = \frac{L P^2}{2 E F} \qquad (18)$$

$$A'' = \int_0^L \frac{M^2 ds}{2 E I} \qquad (19)$$

For an arch of 90°,

$$L = \frac{\pi}{2} r,$$

$$P = w h r,$$

$$F = d,$$

and

$$A' = \frac{\pi}{2} r \frac{w^2 h^2 r^2}{2 E d} = \frac{\pi}{4} \frac{w^2 h^2 r^3}{E d} \qquad (20)$$

Let T from Equation (16) equal the thrust due to shortening of the arch, then the work due to shortening,

$$A'' = \int_0^L \frac{M^2 ds}{2 E I} = \int_0^L \frac{(T y - M a)^2 ds}{2 E I}$$

$$= -2 \int_{45}^{0} \frac{T^2 r^3}{E I} (\cos.^2 \rho \; 1.8 + \cos. \rho + 0.81) \, d \rho$$

$$A'' = 2 \frac{T^2 r^3}{E I} (0.00600) = 15.4 \frac{w^2 h^2 r d}{1 + 9.4 \frac{d^2}{r^2}} \qquad (21)$$

Mr. Mensch.

and the total work

$$A' + A'' = A_1 = \frac{\pi}{4}\ \frac{w^2\ h^2\ r^3}{E\ d}\left[1 + 19.6\frac{d^2}{r^2}\ \frac{1}{1 + 9.4\frac{d^2}{r^2}}\right] \quad (22)$$

For an arch of 120°, the total interior work

$$A_1 = \frac{\pi}{3}\ \frac{w^2\ h^2\ r^3}{E\ d}\left[1 + 4.55\frac{d^2}{r^2}\ \frac{1}{1 + 2.47\frac{d^2}{r^2}}\right] \quad (23)$$

Work of Deformation in a Cantilever Structure.—Assume a slice of the dam 1 ft. wide and h feet high, subjected to full water pressure. The moment about the base is

$$M = wh \times \frac{h}{2} \times \frac{h}{3} = \frac{wh^3}{6} \text{ in foot-pounds,}$$

and, according to Equation (19),

$$A_2 = \int_0^h \frac{M^2\ d\ h}{E\ I} = \int_0^h \frac{w^2\ h^6\ d\ h}{36\ E\ I} = \frac{w^2\ h^7}{252\ E\ I} \quad (24)$$

provided the thickness of the slice is constant from base to top. This is not the case, but it can be easily shown that the value of this work is affected comparatively little by the change of d, because most of the work is performed in the lower part of the cantilever, where the change of thickness is comparatively small. For example, the work performed in the lower fifth of the cantilever is

$$\int_{0.8h}^h \frac{w^2\ h^6\ d\ h}{36\ E\ I} = \frac{w^2\ h^7}{252\ E\ I}[1 - 0.21] = \frac{w^2\ h^7}{252\ E\ I}\ 0.79,$$

or 79% of the total work; hence a comparatively small error is made when we assume I to be constant and write

$$A_2 = \frac{w^2\ h^7}{200\ E\ I} \quad (25)$$

The work performed for the total length of an arch of 90° is

$$\frac{\pi}{2}\ r\ A_2 = \frac{\pi}{2}\ \frac{w^2\ h^7\ r}{200\ E\ I} \quad (26)$$

We will compare this value with the work in pure compression as given by Equation (20), which latter must be multiplied by h to obtain the value for the entire arch.

$$\frac{A_1}{A_2} = \frac{\text{Work of compression}}{\text{Work of cantilever}} = \frac{\pi}{4}\ \frac{\dfrac{w^2\ h^2\ r^3\ h}{E\ d}}{\dfrac{\pi}{400}\ \dfrac{w^2\ h^7\ r}{\dfrac{E\ d^3}{12}}} = 8.33\ \frac{d^2\ r^2}{h^4} \quad (26)$$

For the Huacal Dam, $d' = 13$ ft., $r = 76$ ft., or $r^2 d'^2 = 10^6$, and the work of compression equals the work of the cantilever action when

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the water stands 54 ft. above the base of the dam. When the water stands only 36 ft. above the ground,

$$\frac{A_1}{A_2} = \frac{8.33 \times 10^6}{168 \times 10^4} = 4.96,$$

hence the arch action can take up only 16% of the load on the dam. The value of A_2 of Equation (24) is only correct as long as the resultant forces strike the base within the middle third. Inasmuch as we cannot figure on any tensile stresses in the joint of the dam, quite another distribution of stresses takes place when the resultant comes nearer to the down-stream edge. The joints at the up-stream edge open up, and the compressive stresses will increase in a much greater ratio than the ordinary straight-line formula indicates.

FIG. 24.

Fig. 24 shows that the maximum moment which the Huacal Dam can exert as a cantilever per linear foot equals the weight of the dam multiplied by 0.3 d'. The weight of the dam is about 127 500 lb., and the base is 13 ft., hence the maximum moment at which the resultant strikes the middle third is 510 000 ft-lb. As soon as this moment is exceeded, tensile stresses will appear at the up-stream base of the dam.

In non-reinforced dams the joints cannot be relied on to take the tensile stresses, and, as soon as the foregoing moment is exceeded, cracks will appear and allow the water to exert an upward pressure. In the extreme case, the resultant will strike the base near the down-stream edge, which requires a moment from the horizontal water pressure of approximately $G \times 0.6\, d' = 1\,020\,000$ ft-lb. In this case the joint will be open nearly the entire width of the base, and the upward water pressure will exert a moment of

$$62\tfrac{1}{2} \times h \times d' \times \frac{d'}{2} = 31.25\, h\, d^2 = 525\,000 \text{ ft-lb.}$$

It simply shows that after the resultant strikes the base at the edge of the middle third, the cantilever action in non-reinforced dams is practically exhausted.

The height of water which will exert a moment of 510 000 ft-lb. is found from the following equation:

$$M = \frac{w\, h^3}{6}, \text{ or } h = \sqrt[3]{\frac{6\, M}{60}} = 36 \text{ ft.}$$

It will be assumed that the water stands 36 ft. above the base and that the portion of the dam above this level does not influence the action below.

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If the dam acts as an arch only, the deflection in the center of the dam at the base would be given by Equation (17),

$$\frac{2.028\, w\, h\, r^2}{d'\, E} = 0.00716 \text{ ft.} \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

If the dam acts as a cantilever only, the deflection of the point 36 ft. above the base would be

$$\int_0^h \frac{M\, x\, dx}{E\, I} = \int_0^h \frac{w\, h^3 \,.\, h\, d\, h}{6\, E\, I} = \frac{w\, h^5}{30\, E\, I} = \frac{12}{30}\, \frac{w\, h^5}{E\, d^3}$$

$$= \frac{12}{30}\, \frac{6.25 \times 36^5}{288 \times 10^6 \times 12.833^3} = 0.00256 \text{ ft.} \ldots\ldots\ldots (28)$$

The line, *H M K,* in Fig. 25, shows the ideal deflection of the dam acting as an arch only when the water is 36 ft. above the base, and the line, *G L,* shows the ideal deflection of the dam acting as a cantilever, assuming that the upper portion of the dam is disconnected from the lower portion. Inasmuch as a portion of the water pressure is taken up by the arch action, the deflection of the cantilever must be less, and may be represented by the line, *L' M' G.* At any point, *P,* for example, the total water pressure represented by the deflection, *P O,* is divided into two parts, one which acts on the dam on the arch principle, represented by *P Q,* and one which acts on the dam on the cantilever principle, represented by *Q O.*

We have seen from the foregoing that in a pure arch structure the deflections are proportionate to $\frac{w\, h}{d}$, hence the water pressure at the base is directly proportionate to *G K*; at *P* they are directly proportionate to $\overline{P\, O}\, \frac{d}{d'}$; then that part of the pressure acting on the cantilever principle is directly proportionate to $\overline{Q\, O} \times \frac{d}{d'}$, and that part of the pressure acting on the arch principle is directly proportionate to $\overline{P\, Q}\, \frac{d}{d'}$. The area, *G M' K'*, reduced by the proper $\frac{d}{d'}$, represents the sum of all the pressures acting in a down-stream direction, which try to overturn the dam as a gravity structure, the area, *G H L'*, reduced by the proper $\frac{d}{d'}$, represents the sum of all pressures acting on the arch principle; and the area, *H L' M'*, properly reduced, represents the reaction in the up-stream direction on a slice of the dam which helps to prevent overturning.

It is also clear that the maximum moment at the base of the dam, when the water is 36 ft. above it, is less than 510 000 ft-lb., and,

therefore, at no point of the arch does the resultant of forces come outside of the middle third of any section. It will now be assumed that the water level is 54 ft. above the base of the dam. *H'' M''' K''* represents the deflection of the dam as a pure arch structure. *G M'', L''* would represent the deflection of the dam as a cantilever structure if Mr. Mensch.

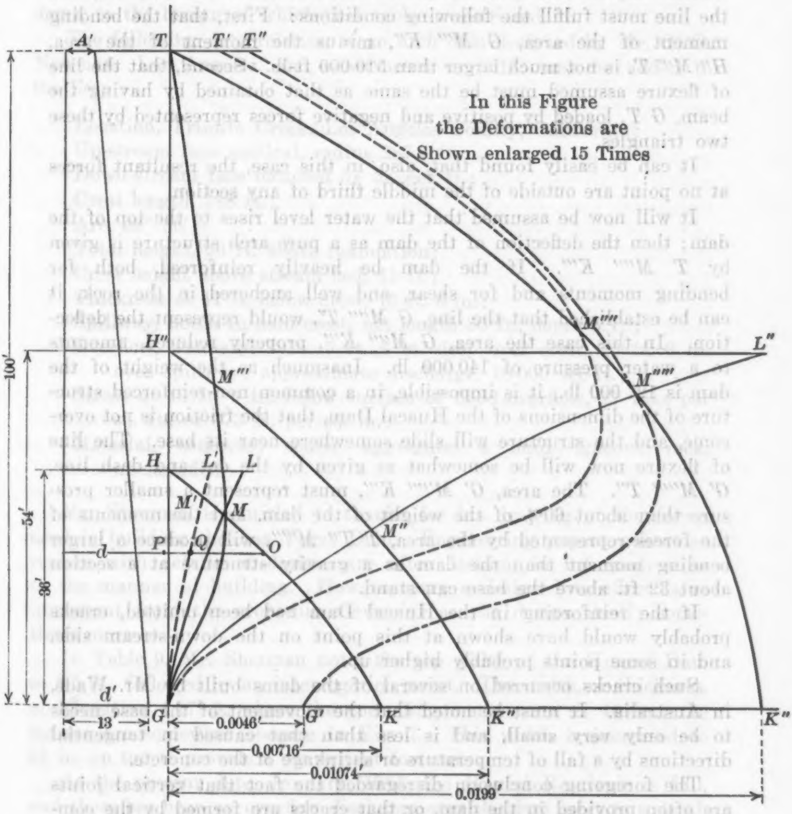

FIG. 25.

a slice of the dam could sustain moments caused by the water pressure of 54 ft., which, however, is not the case, and would be possible only in heavily reinforced dams.

It must be further considered that a slice of the dam does not really act as a pure cantilever; it really acts as a girder supported at

Mr. Mensch. the base against sliding by a horizontal up-stream reaction and by the bending moment of 510 000 ft-lb., and supported in the upper portion of the dam by the flexible support consisting of the top portion of the dam. The line, $T\ M'''\ G$, represents the deflection of the vertical slice of the dam. This, of course, is a mere assumption, and the line must fulfill the following conditions: First, that the bending moment of the area, $G\ M'''\ K''$, minus the moment of the area, $H''\ M'''\ T$, is not much larger than 510 000 ft-lb. Second, that the line of flexure assumed must be the same as that obtained by having the beam, $G\ T$, loaded by positive and negative forces represented by these two triangles.

It can be easily found that, also, in this case, the resultant forces at no point are outside of the middle third of any section.

It will now be assumed that the water level rises to the top of the dam; then the deflection of the dam as a pure arch structure is given by $T\ M''''\ K'''$. If the dam be heavily reinforced, both for bending moments and for shear, and well anchored in the rock, it can be established that the line, $G\ M''''\ T'$, would represent the deflection. In this case the area, $G\ M''''\ K'''$, properly reduced, amounts to a water pressure of 140 000 lb. Inasmuch as the weight of the dam is 127 000 lb., it is impossible, in a common non-reinforced structure of the dimensions of the Huacal Dam, that the friction is not overcome, and the structure will slide somewhere near its base. The line of flexure now will be somewhat as given by the dot and dash line, $G'\ M'''''\ T''$. The area, $G'\ M'''''\ K'''$, must represent a smaller pressure than about 60% of the weight of the dam, and the moments of the forces represented by the area, $T\ T''\ M'''''$, will produce a larger bending moment than the dam as a gravity structure at a section about 32 ft. above the base can stand.

If the reinforcing in the Huacal Dam had been omitted, cracks probably would have shown at this point on the down-stream side, and in some points probably higher up.

Such cracks occurred on several of the dams built by Mr. Wade, in Australia. It must be noted that the movement of the base needs to be only very small, and is less than that caused in tangential directions by a fall of temperature or shrinkage of the concrete.

The foregoing conclusion disregarded the fact that vertical joints are often provided in the dam, or that cracks are formed by the combined action of a drop of temperature and the shrinkage of the concrete when the reservoir is empty.

The result is a considerable increase of the stresses due to the shortening of the arch, and a greater movement of the base of the arch when the reservoir is full. This, however, will not materially change the theory presented by the writer.

H. HAWGOOD,* M. AM. SOC. C. E. (by letter).—The details of the dams at Winchester, Ky., and Lewiston, Idaho, contributed by Mr. Sherman, make a valuable addition to the list of arch dams in Table 1. It does not appear, however, that the other dams cited by Mr. Sherman should be given place in this table, for they are not pure arch dams, to the listing of which that table is restricted. Mr. Hawgood.

The following data relating to an arch dam designed by Mr. G. O. Newman, Los Angeles, and constructed in 1913, may be added to the list:

Location, Triunfo Creek, Los Angeles County, California;
Up-stream face vertical, radius 287.9 ft.;
Down-stream face battered $3\frac{1}{2}$ in. per ft.;
Crest length, 148 ft.;
Arc, 29° 36′;
Total height, 50 ft. above foundation;
Crest height above stream bed, 41 ft.;
Thickness at 41 ft. below crest, 15 ft. 6 in.;
Spillway, notch in dam crest, net length 80 ft., depth 5 ft.;
Overflow, January, 1914, 5 ft. 6 in. deep over crest, 10 ft. 6 in. over spillway; approximate discharge, 10 000 sec-ft.;
Stress at 46 ft. 6 in. below surface of flood overflow, 27 tons per sq. ft., or 373 lb. per sq. in.;
Material, concrete; coarse aggregate, a hard igneous rock, unclassified.

Mr. Sherman is entitled to thanks for his drawing of the forms used in building the Las Vegas Dam. Engineering papers, otherwise replete with details of design, are often lacking in information as to the manner of building. How to carry a design into effect is as important from the economic viewpoint as the design itself; sometimes more so.

In Table 9, Mr. Sherman notes the top width of the Huacal Dam as 3.5 ft., followed by an interrogation point. The width, or thickness, at Elevation 4 179.5, where the corbelling commences, is 44 in.; the figures are somewhat obscure in Fig. 7. This width is increased to 51 in. on top to provide a convenient footway.

At the present date, January, 1915, the Huacal Reservoir is full, and there has been an overflow discharge over the dam crest to a depth of 18 in. The structure is designed for an overflow of 4 ft. 6 in.

The subject of overflow is of more than passing interest, and it may be well to note that the La Grange Diversion Dam, on the Tuolumne River, California, has been subjected to overflow to a depth of 17 ft. The discharge corresponding to this depth is about 80 000

* Los Angeles, Cal.

Mr. Hawgood.

sec-ft. The La Grange Dam, completed in 1894, is reputed to be the highest overflow dam in the United States, if not in the world. The height of the up-stream face is 125 ft., and of the down-stream face 129 ft. It has a gravity section, with ogee profile, and was completed in 1894.

The writer appreciates Mr. O'Shaughnessy's criticism, but does not share with him any apprehensions as to greater risks sustained by an arch than by a gravity dam. Indeed, the arch type has fewer elements of danger than the gravity type. Soundness of foundation, essential for both, is essential to even a greater degree with the gravity than with the arch type. The danger of hydrostatic pressure tending to lift the dam, a serious danger to the gravity type, is a matter of no such moment to the arch. The sliding of a stratified foundation rock, as in the Austin, Tex., dam failure, would not occur under an arch type where the water thrust is transferred to the abutments. The thrusting apart of the flanking hills is inconceivable. The crushing of the arch itself is eliminated by the adoption of safe unit stresses. As to earthquakes, is it not true that a *temblor* sufficiently violent to wreck a structure having some degree of flexibility, such as possessed by an arch, would also wreck an inflexible gravity dam?

Mr. O'Shaughnessy has requested more information regarding the spillway gates. Each of the nine gates is 98 in. wide, with the central pivot ¾ in. to one side of the true center. The narrow leaf has a height of 4 ft. 6 in., and the wider one a height of 2 ft. 9 in. The pressure on the two leaves is in balance when water overflows the lower and wider leaf to a depth of 9 in. A further rise of water will bring excess of pressure on the higher and narrower leaf, and cause the gates to swing open.

The floods of this season were not quite of sufficient height to test fully the operation of the gates. They were successful as far as tried out.

The field of the arch dam is not confined to narrow gorges, witness the Barossa Dam, 472 ft. long, and the Parkes Dam, 540 ft. long. It has also an economic field in the case of a dam too long for the pure arch type by taking the place of the gravity section in the highest, and hence the most costly, part of such a dam, the gravity sections on each side serving as abutments. Multiple arches covering the entire length of the dam could be used in such a case. It would be unduly extending the scope of this paper, however, to take up the question of multiple arches.

During the discussion of the paper* by Mr. L. A. B. Wade, entitled "Concrete and Masonry Dam-Construction in New South Wales", it was brought out that the arch type required about the same volume

* *Minutes of Proceedings*, Inst. C. E., Vol. 178, p. 1.

of masonry as the gravity type when the radius of curvature, in feet, was somewhat more than twenty times the limiting stress, in tons per square foot. This approximation is useful as a preliminary test as to the suitability or otherwise of the arch for any particular location. Mr. Hawgood.

Theoretically, the radius necessitating the minimum quantity of masonry is that which makes the central angle of the arch about 133 degrees. Mr. Jorgensen, in his article, "Arch Dam Design",* goes into the subject at some length.

Mr. Wade expressed the same result by saying that the most economical radius was that which made the versed sine about one-third of the chord.

In a still simpler form, the theoretical economic radius is equal to the chord multiplied by 0.54.

The foregoing will serve to give the maximum and minimum radii within which the best one is to be sought. It will rarely happen in practice that the mathematical formula will control. The configuration of the rock walls which are to serve as abutments, and particularly the relation thereto of the resultant arch thrust, will usually exercise a strong influence on the selection of the radius best adapted to the location.

Replying to Mr. Jorgensen, the vertical reinforcement of old 12-lb. rails was introduced for two reasons: First, to take up temperature stresses, which, under the conditions, may be quite material, because the down-stream face is exposed during by far the greater part of the day to a hot sun, followed by cool nights, and the up-stream face is always in the shade and more or less covered by water. Secondly, the rails formed stout stanchions to which the horizontal reinforcement could be wired most conveniently at the desired spacing, and in other ways they were useful as anchorages during construction operations.

Mr. Mensch, in adhering to the theory of a dam connected to a rigid base, misinterpreted the writer's statement that "Deformation of the rock undoubtedly takes place, and, such being the case, the arch action takes place in some unknown degree from the very base of the dam." Deformation was here used, and obviously so, in the sense of elastic deformation, and not in the sense of dislocation, as interpreted by Mr. Mensch.

The writer is of the belief that there is no substance in Nature, known to engineers, which is absolutely incompressible, inextensible, or void of elastic properties, and, such being the case, it must then follow that the rock on which the dam is built is deformed by the stresses to which it is subjected, just as the same rock, if wrought into the voussoirs of an arch bridge, would contract and expand with

* *Engineering News*, July 25th, 1912.

Mr. Hawgood. every change of load and temperature, as shown by the vertical movement of the arch crown.

Mr. Mensch's treatment of the theoretical stresses in arch dams is a contribution to the mathematics of the subject which will be appreciated by the members. Such arguments are most useful, but their field is suggestive rather than conclusive. The wide difference between theoretical and actual results is strikingly illustrated by comparing the theoretical deflection curves, shown in Fig. 25, with the actual deflections of the Barren Jack Dam, shown in Fig. 6.

One weakness of attempts at mathematical solution is the lack of real knowledge as to the true value of the modulus of elasticity, and the impossibility of one value applying to all parts of a structure changing from concrete thick with plum stones to concrete without any, together with all the variations in manner and perfection of mixing and placing. A recognized value of the modulus for 1:2:4 concrete is 3 000 000 per sq. in., or 432×10^6 per sq. ft. Mr. Mensch uses 288×10^6, and undoubtedly is not without authority for so doing. The value to be adopted is purely a matter of opinion, and the conclusions will be as divergent as the opinions.

Another weakness is the common assumption that the dam is held absolutely rigid at its base. Such an assumption is contrary to fact. The typical foundation of a dam is a trench in the country rock, filled with concrete, on which the dam proper rests and to which it is commonly keyed. The concrete base has essentially the same modulus of elasticity as the dam above it, and will yield proportionately to the thrust brought on it by the water thrust on the dam. The elastic deformation will not be infinitesimal, but appreciable, even if small, and thus the theory of a rigid base is ended. Again, the rock itself is not without elasticity, and when it is considered that most rock, even when structurally strong and sound, is possessed of microscopic seams, the closing of these, under great pressure, together with the compression of the solid parts themselves, may well result in appreciable deformation which is as infinitesimal as Mr. Mensch would have us believe.

Again, the vertical pressure of the dam, as the French engineer, M. H. Billet, pointed out, produces a transverse dilation in the dam and its base.

All these things, together with other unknown conditions, render any mathematical conclusions but conjectural.

The question as a whole is complex. Elements such as the vertical and lateral deformation of the bed-rock above the dam under its burden of superimposed water help to increase the base deformation in the down-stream direction, with opposing forces, on the other hand, tending to counteract. A complete analysis of the subject is impossible, there being too many unknown factors. The writer's object at this time

will have been accomplished if sufficient has been introduced to indicate a reason for thin arch dams not cracking at their bases, as would inevitably take place if the cantilever action were in reality as great as we have been led to suppose on the rigid base theory. Mr. Hawgood.

The theory that the lower part of the dam is held fast in a vise of an absolutely unyielding, rigid nature cannot be logically deemed otherwise than an arbitrary assumption repugnant to the facts.

The theory that the rock base of a dam is deformed within its elastic limits under the stresses brought on it is in harmony with the laws of Nature, so far as known.

In discussing the deflection of the Huacal Dam, Mr. Mensch, in Equation 27, assumes the contained angle of the dam at its base to be 90°; this is not correct, the actual angle is 52°, and the deflection at that point would be about 25% less than that given by Equation 27, and other corrections will ensue.

Mr. Mensch speaks of possible maximum stresses in the Huacal Dam as at least double those ordinarily assumed, and in most of the other dams on the list as from 30 to 60% greater. This statement unqualified might inadvertently create the erroneous impression that the Huacal Dam was subjected to unusual stresses, whereas the contrary is really the case, that is, the Huacal Dam stress is 16 tons $\times$ 2 = 32 tons: that of the other dams is 28 tons $\times$ 1.5 = 42 tons.

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TRANSACTIONS

Paper No. 1321

SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS*

By Clarence T. Johnston, M. Am. Soc. C. E.

With Discussion by Messrs. Herbert E. Bellamy, John H. Lewis, Robert E. Horton, George P. Decker, and Clarence T. Johnston.

Synopsis.

The laws generally applied in the United States in the administration of streams originated in the countries of Northern Europe. These countries developed under the feudal system, in contrast to Egypt, India, and China where, at a very early date, wise principles relating to the administration of streams were discovered and applied in practice.

Northern Italy is a striking example in so far as the development of principles relating to the administration of streams is concerned. Under the dominion of the Roman Empire, wise principles were applied. Spain brought other ideas of a beneficial character. The influence of the countries of Northern Europe, from which we borrow many of our laws and customs, was detrimental to the people of the Valley of the Po.

The doctrine of riparian rights is of feudal origin. It contains no positive principle, and it has outlived its usefulness. It can only be applied by the Courts, and the decisions of our tribunals of justice are not in harmony as to the interpretations which should be given the

* Presented at the meeting of September 2d, 1914.

doctrine according to character of uses and varying climatic conditions. We should not depart from a doctrine which has been followed for many years, until we have something better to suggest in its place.

The first principle that should be recognized is that of public ownership of streams, lakes, and other bodies of surface water. The recognition of this principle lays the foundation for an administration which, without great expense to the various water users, can protect all in proportion to the uses perfected, the character of such use, and with due consideration of all public interests.

Priority in time of use shall give the better right.

The right to use water gives the user no property interest in the water itself, and such right cannot be separated from the use, or transferred to other uses by such user. As long as the user has no personal control of the water supply, he has nothing with which he can speculate to the injury of others.

Preferred uses should be stated in order, in the law, and then protected in proportion to the value of each in preserving the lives of the people and promoting the general prosperity of any particular area.

Beneficial use should be the measure, the limit, and the basis of the right.

A declaration of abandonment should be made when the user fails to apply the water to some beneficial purpose within a reasonably short, specific time.

Even had these principles been recognized by the Courts, it would have been difficult for these tribunals to secure and judge the relative importance of the technical data that must be at hand before a just determination of rights may be made.

A compromise between Court and direct administrative control brings but little reform, because the responsibility is then divided and the defects of both systems are likely to be magnified.

The administration of streams lies within the field of the engineer. The best governed countries of the world, in this respect, have long since provided technical departments which administer this kind of public business in a very satisfactory way. The history of some of the countries of Europe is being repeated in America, and the engineer should be ready to accept the responsibility that, sooner or later, must come to him.

The manners and customs of the countries of Northern Europe have been introduced in the United States and accepted quite generally by the people. The climate and topography of the eastern half of the American Continent compare favorably with those of England, France, and Germany. It is natural, possibly, that the rules applied in the administration of streams during the early history of America should be similar to the regulations enforced on the Thames, the Seine, and the Rhine. Where irrigation is essential to agriculture, we find an early interest manifested in problems relating to the diversion and use of water. The influences of the countries of Northern Europe in this particular would be more healthful if irrigation were practised there. It is probably unfortunate that the people of the United States have not been taught, by some early experience, that streams should be considered public property and one of the great natural resources.

In the older, irrigated districts of the world we find some good examples of wise laws relating to the administration of streams. China and India can teach us many lessons. The modern system adopted by Australia merits our study. Egypt, that land which has never enjoyed freedom, and has always been a victim of war, famine, and pestilence, administers its one river, the Nile, in a manner which can only call for praise from citizens of countries which boast of a higher type of civilization. Although the rulers of Egypt, both foreign and domestic, have robbed the people of their lands, have taxed them to the extreme limit, have compelled them to labor for the public without compensation, and by centuries of oppression have destroyed their initiative and broken their spirit, yet the Nile has always been public property. It is the great highway of commerce. It is the life of agriculture and the foundation of every other industry. The Nile is administered by public officers for the general benefit of the people. No private rights, which tend to threaten the prosperity of the people, are recognized or protected. The principles that have been accepted by Egypt are fundamental in their character, and they are applied uniformly throughout the extent of the Nile Valley. There is no conflict between farmers under the same canal, between communities depending on different canals, or between adjoining provinces. The principles that are essential are so simple that the native can understand and appreciate them. This immediately brings about a feeling of mutual confidence and respect between the water user and the officer in charge

of the local administration. Finally, there is no litigation in Egypt between users of water, whether they be individual farmers, corporations, or municipalities.

The laws of Imperial Rome, relating to the administration of streams, furnish examples which are superior to those we have introduced from the countries of Northern Europe. These laws were based on the idea of public ownership of streams. They recognized that water must be diverted from the natural channel of the stream, in many cases, if beneficial uses are to be served and protected. They gave the builders of canals and other works the right to condemn private property. Those building improvements of this kind were obliged to respect the rights of others and to compensate those whose property was injured.

Northern Italy, like Egypt, has been a battle ground where the nations of the world have staged many important campaigns. Spain, France, and other nations have in turn dominated portions of the Valley of the Po. The influence of these invaders has been felt long after they surrendered political control. It would be assumed that the influence of Spain would not be detrimental in so far as the administration of streams was concerned. This was demonstrated. That country had gone through an experience that was profitable. The invasion of the Moors was beneficial to Spain in this respect, because these people from Northern Africa had been reared in districts where public control of the water supply was essential to life. The Spanish influence in Northern Italy, therefore, was salutary.

The influence of the people of France and Germany was detrimental. The countries of Northern Europe had developed under the feudal system. Because it was not necessary to divert large volumes of water from the streams to support the population, safe and sane principles were not evolved at home, and no apparent effort was made to discover and introduce rules that had been applied in countries which had experienced healthy, early development. The feudal countries always emphasized the importance of protecting private property for the benefit of the few owners thereof. The countries of Southern Europe and Northern Africa, on the other hand, recognized at an early date the necessity of public supervision of streams for the benefit of the many. When a portion of the Valley of the Po fell under the dominion of France, principles of feudal origin were intro-

duced. Concessions were made by the State and then by the Church. The fortunate individuals who received these concessions became owners of specific volumes of water, and speculation became general. The principles of the original Roman law were broken down here and there, and much confusion resulted. Litigation soon began, and many of these cases remained in the Courts in some form for several hundreds of years. Not until 1884 did Italy recover from this influence, in so far as its laws were concerned, and it will be many years before it recovers fully. Some plan must be formulated, and then carried into effect, whereby the old concession may be canceled. While Northern Italy was throwing off the customs it had inherited through early years of feudal association, some of our Western States were passing through a similar experience. The reforms that have taken place there and here are quite similar, yet neither country has profited by the experience of the other.

As the customs of Northern Europe, rather than those of other countries, were first introduced into the United States, we are somewhat concerned in their origin, the principles they embraced, and the method in which they were applied. As streams were of only passing importance to the feudal lords, and as private property was not held by the masses, it was probably apparent to them that a single rule might answer for all time. It was plain that some rule should be applied which would prevent the pollution of streams. As streams were of some importance to navigation, and as the owners of land bordering the watercourses desired stable conditions, in so far as discharge was concerned, it did not seem wise to encourage diversions from the rivers which would materially affect the flow thereof. The rule which gradually developed is approximately as follows: All persons owning lands abutting on a natural stream have the right to demand that the waters of that stream shall pass their lands undefiled in quality and undiminished in quantity. We call this the doctrine of riparian rights.

If this rule is carried into effect rigidly, water cannot be diverted from a stream in any way. The thirsty laborer who drinks from a convenient brook must diminish the "quantity" of water flowing therein. Streams must become more or less "defiled" with every contact with animal life. It was a theoretical rule, in the first place, this doctrine of riparian rights. As it had its birth in oppression and

despotism, so its application to the streams of our time has brought political and industrial slavery and injustice. The doctrine is a part of the unwritten law of England. Because it is theoretical in character and based on no well-defined working principles, it has furnished but little light to the Courts of our day.

The doctrine was introduced in the United States with the common law, and applied by our Courts, not by our law-makers. Its application has extended by judicial decree, so that its interpretations are only limited by the number of our Courts. It is seldom recognized and never defined by State or Federal statute. It was unjust to the Courts to thrust upon them the responsibility of protecting water users. When the law does not provide an administrative department for the purpose of transacting public business, those who would otherwise be protected must war among themselves and finally resort to tribunals of justice for relief.

We need no evidence, other than that contributed by the Courts themselves, to prove that our present methods of stream control, based on the doctrine of riparian rights, are not supported by principles that are sound. Every city, every water-power plant, every individual who resorts to the stream for domestic water supply, is violating the doctrine of riparian rights, yet this doctrine prevails almost alone east of the Missouri River. To be sure, the Courts hold that if the doctrine of riparian rights has been violated for a term of years the user may obtain a specific volume of water under what they call a prescriptive right.

Although this may give relief, in certain cases, yet it goes to demonstrate the weakness of the parent doctrine. It is an indication that the Courts recognize that some other rule must apply, if very obvious injustice is to be provided against.

The early decisions of the Supreme Court of Michigan hold that an owner of land abutting on a watercourse also owns the land of the bed of the channel to the "thread" of the stream. This term "thread" may be defined in theory, but presents some difficulties in practice. The Supreme Court of Michigan intimated, at first, that this was to be a general rule. It was to apply, and possibly has applied, to navigable streams as well as to those that are not navigable. Under this theory, the citizens of Detroit own the bed of the Detroit River to the "thread" of the stream. The same theory was stretched a little when

applied to the riparian lands bordering Lake St. Clair. It applied again to the St. Clair River, but when Lake Huron was reached the Court finally had to admit that the riparian owner had to limit his possessions at the meander survey of the lake. This would seem to show that a fundamental error had been made in the early decisions. The meander line should be considered the boundary between private and public property throughout. No private citizen can exercise exclusive control of the bed of a river without interfering with public interests. The doctrine of riparian rights, without modification, has but little to do with the protection of legitimate uses or the rights of the people at large. The Court, having no other rule to govern its acts, was compelled to construct its decisions with the timber at hand.

The writer has studied the application of the doctrine of riparian rights in countries of three continents. He has followed the history of its introduction into the United States, and watched its application as settlement has taken place, and as some kind of stream control has become necessary. He has found no two authorities who entertain the same views as to the theory of the doctrine, or as to its limitations in practice. If the doctrine has any value, if there are reasons why land owners along streams should have control of such streams, engineers, by this time, must have satisfied themselves as to the justification therefor. A discussion of the doctrine by engineers may be considered by some as an invasion of the field of the attorney. This is not true. It is not necessary to attempt to understand the intricacies of the theories that have been developed by the Courts. As the doctrine has been applied in the United States for more than 100 years, it should have demonstrated something as to its worth, and if it embraces a principle, this should be manifest to the layman. If the principles on which it is based are so obscure that they can only be understood and appreciated by the judiciary, then the doctrine should be abrogated, and something more tangible should be substituted in its place.

If the doctrine, such as it is, merits a stay of sentence, the engineer, who must always stand between the Court and the water user, should demand that its meaning be accurately and concisely defined, and its limitations be absolutely established.

The doctrine of riparian rights does intimate that the public (somewhat limited) is concerned in every stream. Court decisions, based

thereon, generally concur in the principle that the public owns the water, and only the use is acquired by private parties. The doctrine of riparian rights attaches the use to the lands owned by the riparian proprietor. A complete revolution is not necessary when we permit actual diversions of water from streams and place all rivers and their tributaries under the supervision of public officers. We simply pass from the application of a doctrine of negative principles to something positive and tangible. We may go back and study the application of the doctrine of riparian rights in other countries and satisfy ourselves as to the wisdom of affording it continued support here. When Great Britain enters new territory she does not apply the common law, to the detriment of her subjects. The doctrine of riparian rights does not apply in Egypt, South Africa, India, or Australia, as it does here. There, the Courts have never been compelled to make laws governing any resource.

Fortunately, we have some examples of better things, even in the United States. A few of our Western States have provided engineering administrations which, not only have custody of the records relating to the initiation of rights, but also determine the relative rights of all claimants. In order that such administration may do justice to private and public rights, principles are defined in the statutes that govern these departments. There, a land owner has no rights in a stream simply because his property is located along that stream. The great aim of the law is to encourage uses which develop the country, which establish homes, and, at the same time, preserve all public interests.

Before we discard a rule that has been applied for many years, we should have something better to substitute in its place. The administration of streams is very complex and difficult unless the few, wise, fundamental principles, discovered and enforced many years ago, are recognized. Though not assuming to refer to all principles that might apply, the writer will discuss a few of the more essential rules somewhat in the order of their importance. As has been intimated, the principle of public ownership of streams is the first essential to a just administration of the water supply.

a.—All streams, lakes, and other bodies of water, within the exterior boundaries of any country, should be, and always remain, public property. This means that the stream bed and the water flowing therein should be thus considered. There are many reasons why this prin-

ciple should be upheld. There is a physical reason, which is seldom considered. Streams are naturally of a public character, in so far as they are distinct from political lines. A stream, therefore, is a matter of general public concern. In addition, water cannot be privately owned. If Thothmes I had upheld the theory that private ownership of water is possible, and, to illustrate his ideas, had bottled a quantity and dedicated it to the generations of his family that followed him, the receptacle only would remain to those representing his house at the present time. Water is constantly moving. Like air, it is controlled by laws which were framed by a Higher Authority. We may enjoy the use, but not the possession.

As soon as streams, or any surface waters, become commodities for trade and speculation, the local public, responsible for such a condition, is openly defying natural laws and thereby imposing burdens on those who cannot protect themselves. No act of a despotic or careless government will work greater injury to its citizens than to permit, by decree or sanction of law, the private ownership of water. One might conceive of a government which would encourage a private monopoly of the air. It is at once apparent that this would be disastrous. Water falls in the same category, but because a profitable traffic can be carried on with apparent success, for a limited time, we are not shown, at once, the iniquity thereof. Though private parties have been able, frequently, to speculate in water, yet it is probable that the public has lost none of its inherent rights, and that both the buyer and seller have been deceived as to the real effect of the deal when consummated.

Public control of streams results in the determination of claims without great cost to water users. Where the Courts have jurisdiction, only the claimants who are financially able to maintain themselves in a controversy (that generally continues for years) can take an active part in the "adjudication" proceedings. Under the best administrative systems, the water user has no expense except that incurred in the payment of a nominal fee of two or three dollars. The public makes the surveys and measurements required, and collects all field data. The tabulation of rights is prepared by public officers, and hearings are provided where all users interested may protest against statements made as to the time water was used for beneficial purposes, and similar testimony. When the adjudication is accomplished, the final order is carried into effect by the same administration. All water users, con-

cerned in diversions from any stream and its tributaries, are brought together in the single proceeding, and no one, because of financial ability, or personal or political influence, has an advantage. High-class, public service of this kind is being performed in several of our Western States. The older administrations have had an experience of more than 20 years, and we can say now, with some assurance, that these are, in every way, a success. There, we find constitutional and statutory provisions which declare the waters of the States to be public, or the property of the State. These declarations relate to principles, and not to doctrines that have unlimited elasticity in interpretation.

b.—Priority in time of use shall give the better right. That is, the first user of the waters of a stream has a better right than any subsequent user, and the second user has a right superior to the third and those who follow, and so on. It is, perhaps, unnecessary for this principle to be discussed. It is absolutely necessary to make some such rule, particularly as between a large number of claimants concerned in a use of the same kind. It is manifest to one who gives the matter due consideration that, after the early users have made improvements and perhaps established homes, they should be protected as against later comers, who know what has taken place at the time of their appearance. This principle appeals with more force to an administrative officer, perhaps, than it does to one who has never participated in the practical work that falls to an engineer under such a system.

c.—The right to use water gives the user no property right in the water itself; such right belongs to the use and not to the user. This principle recognizes perpetual public ownership of water. The user is protected thereby, because he then belongs to a large class, all of whom are placed on the same basis, and no one person can impose on his neighbors. As long as the user has no personal control of the water supply, he has nothing with which he can speculate, to the injury of others. Although circumscribed by rigid restrictions, the legitimate user is protected, in so far as his use justifies protection, and, at the same time, he is given no special privileges.

Certain engineers framed a water law for one Western State some 20 years ago. They tried to have the measure specific in attaching the rights to use water for irrigation purposes to the land reclaimed, and to make it impossible for the owner of the land to dispose of such water right, except with the land. After some active campaigning

the bill was presented, and the framers were confident that this principle had been incorporated. They had a part in the administration of the law, and this principle was carried into effect. Within a short time an irrigator attempted to make a transfer of his water right separate from his land. Litigation began at once. This controversy in the Court was continued in an active manner for 10 or 12 years. At the end of that time the litigants had lost their farms, these having been consumed in Court costs and attorneys' fees. Finally, the Court decided that the law did not embrace the principle, and the transfer was thereby sanctioned. Those interested directly and indirectly in speculation in water joined in open hostility against any measure which had for its object the correction of the original law for the purpose of incorporating the principle therein. The friends of reform carried on a campaign for 6 years, and their efforts were finally rewarded by the enactment of adequate legislation.

To those who have observed problems of stream control from some distance, this principle may seem to be of but little importance. Wherever it has not been carried into effect extensive litigation has resulted and great injury has been done to water users who were not in a position to protect themselves. The argument made by those who oppose the principle is that any one who is damaged can secure redress in the Courts. This plan has but little to recommend it, because the weaker element is immediately eliminated from the contest. Further, the effect of a transfer, separate from the original use, is seldom apparent at the time of the transaction, and before those who are injured can appreciate the resulting damage and take steps to protect themselves, much money may have been spent in perfecting the new use. There is one exception to this general rule, however. It is often necessary to transfer rights to preferred uses. This should always be accomplished under public supervision. Should a city need water that had been formerly used for irrigation, power, or for other less important purposes, a procedure should be prescribed whereby a transfer of right to use (not a right to the water itself) may be made to the municipality in a public manner, so that all concerned may make their objections at the time, and, if injury results that cannot be foreseen, compensation can be made therefor under the direction of the administrative officers. Water cannot be maintained as public property when private parties sell water rights separate from the use to which they have been

dedicated, except under a public procedure and under the supervision and advice of public officers, so that, in carrying this last principle into effect, we are upholding another of equal importance.

d.—From the early records we are able to read it would seem that there has always smouldered in the breast of man, a feeling that all rights to use water cannot be classified in the same list. The first use that comes to mind relates to the domestic supply—water for drinking and household purposes. We must protect such uses, regardless of all other claims. It is plain that there should be preferred uses. There may be some room for debate as to the exact order in which these should be guaranteed protection. Climatic conditions, the nature of the water supply, and many other items enter as factors in such an undertaking, but an approximate estimate, which is of general application, can be made. No one will deny the necessity for protecting the individual supply for domestic purposes first. This is almost identical with municipal uses. Because many people, combined within the limits of a town, require more water than does any single claimant, the people of towns are often imposed on. Generally, under the doctrine of riparian rights, a farmer or other land owner is justified in taking water from a stream, adjoining his lands, for domestic purposes. As a rule, no effort is made to limit this use. We are compelled to suppose that it must be "reasonable". There seems to be no reason to presume that a single family, within the limits of a town, requires more water than does a farmer, living beyond its borders. Bearing in mind that the uses in both cases are equally high, and the only difference is one of local government, the question arises: When shall we deny the user within the town the same rights and privileges that are granted to the farmer? The Courts would probably justify a decision which discriminates between the two uses by the fact that the farmer is an owner of riparian lands and the city user is not. Assume that each user within the limits of the town is given a deed to a narrow strip of land connecting his property with the river bank. This will be an instance where conditions within the town may be made to fit a fine-haired theory. What, then, will be the status of the city user? He is a riparian owner in fact and a preferred user. Because he and his neighbors join together, and by combined efforts build a water system that will supply all within the limits of the city, there is no reason that we should assume that the character of the use has changed in any

way. The total demand on the stream may increase because of the growth of the town. The character of the use remains the same. Regardless of this, cities and towns have often been compelled to undergo many hardships because the Courts have generally given private enterprise the advantage wherever the two classes of claims or rights have been in conflict. As soon as a study of preferred uses is undertaken, we must begin to readjust some of our former views. If we say that the first user, as to time when use began, has the first right, should this principle apply to uses of a preferred character? When a preferred use expands and becomes a detriment to a use not preferred, what shall be done in order that the higher use may have protection and the lower use be not damaged or even confiscated? In some States, the preferred users are always protected, regardless of the effect thereof on others. This is probably a wise rule when all users understand the situation from the beginning, and when all investors know that the principle will be carried into effect. Probably the better plan is to have an understanding, with users not preferred, to the effect that as the preferred use grows, the inferior users shall relinquish certain rights and be compensated therefor at a fixed rate, regardless of the time the readjustment may take place, or the demands that may be made on the stream by the preferred user. This should all be done under public supervision. Where water is wasted during times when the supply is inadequate for all, the public should interfere. A city should be permitted to expand its use without being compelled to compensate a user who wastes water. Waste, during times of scarcity, should never occur, under any well-regulated system of public control. By waste is meant a loss of water through carelessness, or such loss as may be prevented by a reasonable expenditure of money in repairs or improvements. There are losses which cannot be prevented, and these we cannot well consider.

Where irrigation is essential to agriculture, the use of water therefor should be preferred, to a limited extent at least. Power can be developed in other ways, and power plants may be located where they will not interfere with the use of water for irrigation. The ideal relation of these uses, geographically, would require the power plants to be located up stream from the irrigation works. For instance, in a mountainous country, the greatest fall is obtained near the headwaters of the stream. This is favorable to water-power plants. If a

reservoir can be constructed on a main stream to equalize the flow of water and produce a constant head for the power plant, which then discharges into a second reservoir where the water may be stored until needed for irrigation, no interference will result. It is impossible to maintain a power plant at maximum efficiency on a reservoir where water is stored for irrigation. Irrigation demands the maximum use of water in a short period during the summer, generally speaking. The maximum demand for power often comes during the winter. The head available, at the reservoir which stores water for irrigation purposes, is at a minimum at the close of that season. The supply of water, therefore, would be irregular for the power plant, and the head would not be uniform. Some of the most disastrous litigation in foreign countries, where irrigation is a necessity, has been brought about by the establishment of power plants which retard or prevent agricultural development.

That preferred uses should be recognized and protected, all must admit. When all facts relating to uses on a single stream are brought together, the administrative officer can determine the relative rights based on other considerations; after which, giving these higher uses appropriate weight, the final adjustment can be made. It would seem difficult to balance these rights, keeping two principles in mind at the same time, but in practice this is not as troublesome as it is in theory. Preferred uses are always comparatively small, and the public generally consents whenever a public authority makes a decision which protects them.

e.—Beneficial use, and the extent thereof, at the time the determination of rights takes place, shall be the measure, the limit, and the basis of the right. Where rights and claims are in an inceptive period, the public record relating to the determination may properly mention them, and all prospective users be given a reasonable time to complete the application of water, under the plans submitted to the office of public record. These incomplete rights should be determined later, as the water is applied to beneficial purposes.

The principle of beneficial use is an important one. When it is applied, every user is restricted to a specific volume, as a maximum, and he can only use so much thereof as can be applied beneficially. Under the Court decisions of the country, generally, claimants are given specific allotments of water, and in many places they are per-

mitted to sell what they find they do not need. This practice breaks down nearly every important principle relating to stream administration. The resulting condition is almost impossible, from the standpoint of the water user. Under the principle of beneficial use, a power plant, for example, is not entitled to the uninterrupted flow of the stream, regardless of the needs of the plant. Under the doctrine of riparian rights, as it is interpreted frequently, a water-power plant can require the entire flow of a stream to reach the plant, regardless of how much thereof can be used beneficially. This permits the plant to waste as much as it pleases, and to bring damage suits, even when it is not injured by users above. If the right of the power plant is limited by the volume it can use beneficially, the owners thereof have no control of or interest in the volume that the stream furnishes in excess of the capacity of such plant.

f.—A declaration of abandonment of rights to use water must be made where water is not used for a certain specified period. The application of this principle makes it impossible for those having water rights simply to hold them without using the water. This rule is incorporated in the statutes of many Western States, yet its practical application is always hampered, because the administrative officers do not generally exercise control, and it is indeed seldom that public, rather than private, interests are upheld. We have a network of theories relating to abandonments, and though these are interesting to study within doors, they are worth but little in the field. The Courts have held almost unanimously that "to constitute abandonment there must be a concurrence of act and intent, the relinquishment of possession, and the intent not to resume it for beneficial use, so that abandonment is always voluntary, and a question of fact".* This means that a user, who has been accorded a right to apply water to some beneficial purpose, may cease all use for as long a period as he may desire, and the actual abandonment will only take place when he admits the same. This is a poor rule. When the use ceases for a limited period, this should be taken as evidence that the water right is of but little value to the former user, and he should be deprived thereof. The public is concerned in use, not in speculation. A definite period, quite brief, should be fixed by law for the interval of non-use. Where the use of

* Wiel. "Water Rights in the Western States."

water is suspended, for such a time, as a matter of convenience rather than of necessity, the right should be canceled, and those who can apply the same beneficially should be favored. The application of this principle emphasizes the principle of public control, so that it has a secondary as well as a primary value.

There are other minor principles that might be discussed. The more important general principles have been already referred to, and a law, State or National, which embodies them, will result in an equitable administration of streams. These principles are not difficult to understand. Injustice is always done when they are obscured by statutes which relate only to procedure, and Court decisions that deal with fine-haired theories. It may be said that, had these wise principles been first incorporated in the law, the Courts might have been able to "adjudicate" the various rights in a satisfactory way. The West has tested this procedure thoroughly. All such trials have resulted in failure. Reference will be made to a single Court decision, made after the doctrine of riparian rights had been abrogated. Where irrigation is practised, a large number of claimants are generally represented. Table 1 is given, without referring to any particular State or stream, yet the data are taken from the Court records. Instead of giving the names of the water users, or the ditches, the writer will simply list the adjudicated rights in order of "priority" numbers.

The adjudication was made by a judge who took pride in his ability to frame water-right decisions. If this decree was based on any principle, the latter has been carefully concealed. An examination of the table will show that there is no relation between the area irrigated and the quantity of water allotted. It would seem that the Court permitted all kinds of testimony to be given. That a large part of this was erroneous, has been demonstrated by later investigations. Surveys have shown that the areas claimed to have been irrigated are inaccurate in practically every case. In an action of this kind in the Court, the facts are often gradually concealed by a record which contains quibbles of attorneys, statements of witnesses that are valueless, rulings and motions that are to no purpose, and testimony which has no bearing on the questions to be determined. Often the record becomes so voluminous and obscure that the Court is afraid of it, and the case is indefinitely delayed.

TABLE 1.

(1) Priority number.	(2) Acres irrigated.	(3) Allotment of water, in cubic feet per second.	(4) Area, per cubic foot per second (Column 2 ÷ Column 3).
1	160	8.65	18.4
2	1 200	67.03	17.9
3	800	30.00	26.7
4	150	12.20	12.5
5	800	37.40	21.4
6	400	22.32	17.9
7	3 300	33.55	99.0
8	300	28.00	10.7
9	300	8.40	35.7
10	200	17.90	11.2
11	300	2.35	127.0
12	300	2.25	133.0
13	160	3.6	44.5
14	......	82.0	......
15	600	12.32	48.7
16	160	3.60	44.8
17	1 500	26.50	56.6
18	1 160	84.45	13.7
19	21 000	210.00	100.0
20	800	25.00	32.0
21	2 000	60.10	32.2
22	3 000	78.11	38.4
23	100	10.50	9.5
24	160	17.50	9.2
25	800	12.00	66.8
26	4 000	45.72	89.1
27	1 200	25.00	48.0
28	......	13.00	......
29	100	10.00	10.0

In another case a State made an attempt to compromise between public control of streams, through an engineering administration, and Court supervision. Under this plan, the engineering administration is to obtain the facts and the Court is to make all decisions. This compromise was not accepted by the engineers leading in the movement without protest, but it was the best that could be obtained. This law has been tried out. The facts were determined on a single stream and all its tributaries. Complete tabulations and other needed information were submitted to the Court. This basic testimony has been in the hands of the Court for some 7 years, and nothing has been done. It is not too late, fortunately, for the public to provide administrative officers to take charge of this important work. The change can take place without a revolution, except in methods. Many countries and some of our most progressive States have already taken this step. The Courts have not been offended. On the other hand, prominent jurists have assisted in the organization of such administrative systems.

Within the limits of a municipality, where public rights and public interests are always more or less apparent, we unconsciously acquiesce

in the application of many of the principles discussed in this paper. We admit that the city owns and operates the water system. This is public control of the water supply within the city limits. An individual water user cannot sell water to his neighbor. The individual cannot say that his right to use is limited only by the volume of water that may be discharged on his premises or adjoining property. Waste is not permitted where it can be controlled. The parallel might be carried farther. We find some of the principles recognized in national and international laws relating to navigation.

Laws should be framed for the purpose of carrying principles into effect. Health laws generally define principles. When an epidemic threatens a community, the laws give the health officers authority to act at once. They may interfere with the liberty and the pursuit of happiness of many people, yet we hear but little complaint. Principles are being applied, and their value is generally recognized. As a rule, experts in science, rather than in law, frame these statutes and, in a large measure, interpret them. They know what principles should govern, and the public is willing to permit these specialists to lead, provided the important, fundamental principles be upheld. In this work, the law is incidental. The principles embraced therein are of prime importance, and we seldom question any reasonable interpretation of the law. We are thankful that public health can be protected without the necessity of preliminary litigation. We are thankful that these officers can act without great danger of an injunction being served at a critical time.

Engineers, who are placed in responsible charge of streams, belong to the same category. In fact, they must co-operate with the health service and with other administrative officers. The law, to them, consists of public directions as to their duties and, above all, plain definitions of the governing principles.

The administration of streams lies essentially within the field of the engineer. The engineer understands and appreciates the physical problems that are encountered along every watercourse. He must measure the discharge of the streams and the capacity of the diversion works. He must determine the value of water-power plants and ascertain the area and value of irrigated lands. He must estimate the volumes of water that municipalities require and appreciate the relation that should exist among these various uses.

Having all field data before him, and understanding the history of governments which have solved the problems presented, he should be able to frame laws defining the principles which should apply. These laws should be brief, and the principles should be defined in such terms as to leave no doubt as to the meaning of the phrases used. The principles should be stated first, because there is no excuse for an administration until these have been selected and defined. The officers provided in the statute then have a purpose. The foundation for an organization and for just compensation has been laid. No relief has come in any country until those understanding the physical problems have led in a campaign for laws embracing essential principles. The history of the Old World is being repeated in the New. The engineer receives but little financial reward for service of this kind. It is possible that he may secure more remunerative employment under a system which gives no protection to legitimate water users. The Profession, as a body, has never avoided a plain duty because of mercenary motives. Our great engineers are loved and remembered for the service they have performed, not because of the compensation they have received.

DISCUSSION

HERBERT E. BELLAMY,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with particular interest, not only on account of his familiarity with the subject, having dealt with questions of riparian rights in England, and of his knowledge of the systems in vogue throughout Australia, but because of the comprehensive and lucid manner in which the author has dealt with the whole subject. Indeed, the paper embraces practically every phase relating to the betterment in the administration of streams, so that there is very little, if anything at all, to add. The preamble presents amending principles suggested for adoption impartially, and with which, in every case, the writer cordially agrees. The paper brings together, in a small compass, a great mass of information dealing with the application of the doctrine of riparian rights in many countries, which cannot fail to be of the greatest value in the discussion of the subject by engineers. Mr. Bellamy.

Some years ago, when the writer was investigating some so-called riparian rights in connection with a proposed new water supply for a town in England, he was greatly puzzled to know what these rights were. Decisions of the Courts did not help him, because, as the author remarks, if the doctrine of riparian rights were carried into effect rigidly, water could not be diverted from a stream in any way. Several decisions could be quoted to support this latter contention if necessary, but the following brief abstract from a well-known case merely serves to show that something more tangible should be substituted for the absurd and diabolical doctrine, as it exists in many places to-day. In the case, Mason *v.* Hill, it was decided as follows:

"A riparian proprietor can have no larger right than he has by nature against those above and below him. Hence the right to have a stream to flow in its natural state without diminution or alteration is an incident to the property in the land through which it passes * * *."

The author points out that the first principle which should be recognized is that of public ownership of streams, lakes, and other bodies of surface water. The writer would also include "all underground supplies." This recognition is the fundamental principle of success of stream administration throughout the Australian States. In the writer's opinion, the second principle, *b*, which the author does not discuss at great length, is the positive safeguarding of the interests of the first user of the waters of a stream as against later comers, even including State or National rights.

As the author mentions that the modern system adopted by Australia merits study, the writer submits the following brief digest of

* Sydney, Australia.

Mr. Bellamy. the Water Rights Act of 1912 which is in force in the State of New South Wales.

The Water Rights Act came into force in 1896 and was consolidated in 1902. On November 26th, 1912, assent was given to an Act consolidating the separate Acts relating to Water Rights, Water and Drainage, Drainage Promotion, and Artesian Wells. This Act is administered by the Minister for Public Works. On the passing of the original Act, the right to the use and control of all streams, whether perennial or intermittent, flowing in a natural channel, through or past the land of two or more occupiers, and of all lakes, swamps, lagoons, or other sheets of still water, whether permanent or temporary, situate within or fronting the land of two or more occupiers, passed from the owners or occupiers of the said land and became vested in the Crown; subject, however, to the reservation that the said occupiers shall have the right to use the water on their frontage for domestic or stock purposes, or for gardens up to 5 acres in extent, without the necessity for obtaining a license for any work used solely for those purposes.

Rights granted under the Mining Acts on any public or private statute, are also preserved.

Section 8 provides for the rights of the Crown in respect of works, and Section 9 for the rights of the occupiers to which the Act applies.

The definition of "Work" is comprehensive, and includes any dam, lock, reservoir, weir, flume, race, channel (whether an artificial channel or a natural channel artificially improved), any cutting, tunnel, pipe, sewer, and any machinery and appliances; and "Work to which this part extends" means work connected with any river or lake flowing through, or past, or situate within the land of two or more occupiers, or with any water flowing in, to, or from, or being in any river or lake flowing or situate as aforesaid, whether such work be for water conservation, irrigation, water supply, or drainage, and whether such work be constructed before or after the commencement of this Act.

Section 10, and succeeding sections, provide for the granting of licenses to private individuals. Any person being in actual occupation of the site of a proposed work may make application for a license. The application should be forwarded to the Minister for Public Works, who will then advertise its receipt, together with a date and place at which a public inquiry will be held as to the advisability of granting the application. These inquiries are usually held by the local Land Board, and any person affected by the application may attend and give evidence for or against the granting of the application. Any person aggrieved by the finding of the Board may appeal to the Land Appeal Court within 28 days of the date of such finding. The finding of the Board is published in the *Government Gazette*, and, after the

expiration of 30 days from the date of publication, the Minister, where the Board recommends the granting of the application, and provided no appeal is pending, shall issue a license for such period, not exceeding 10 years, as he may think fit, subject only to such conditions, if any, as may be embodied in the Board's finding. The applicant is notified by the Minister of the term for which it is intended to grant a license, and the amount of the fee which must be paid before a license can be issued. Mr. Bellamy.

Section 13 of the Act provides for joint application by two or more occupiers, as, for instance, persons on opposite sides of a stream who desire to construct a dam.

Section 17 is very important, because it secures to the holder of a license quiet enjoyment, and the sole and exclusive use, of the licensed work, as against all persons, including the Crown.

Section 18 provides for a penalty for any alteration of work during currency of a license.

Sections 19 and 20 provide for the carrying out of works by the Crown, the formation of "districts" benefited by such works, the assessment of charges in each and every case for benefits accruing to the properties within the "district", and the payment of charges by the persons benefited. The actual procedure is conducted as follows:

"The Governor notifies by proclamation in the *Government Gazette* a proposal for the construction of a dam, lock, weir, channel or drainage work, as the case may be, to be constructed by the Crown, together with an estimate of cost. This notification is followed by the gazettal of the boundaries of a 'district,' the land within which will, in the opinion of the Local Land Board, be benefited by the construction of the proposed work, and within which water or drainage charges may be levied. It is then necessary that a 'two-thirds majority' of the occupiers, who must also occupy an area exceeding two-thirds of the total area within the proclaimed district, petition the Land Board, on the proper form, to carry out the proposed work. In default of this petition the proposal lapses. On receipt of the petition, the Land Board may report to the Minister, recommending that the work be carried out, and after the expiration of thirty days from receipt of the Board's recommendation, the Minister may carry out the work with funds legally available. On completion of the work, the Land Board, by direction of the Minister, assesses the charges to be paid by each individual occupier, and the aggregate of these charges must not exceed six per cent. on the total cost of the work. The charges commence from the date of completion of the work, which must be advertised in the *Government Gazette*. The assessment takes place in open court, and the persons affected may attend and give evidence on their own behalf. On the petition of persons liable to pay one quarter of the total assessment, the Land Board must make a fresh assessment."

Mr. Bellamy.

The remaining Sections, 21 to 27, refer to miscellaneous matters such as injuries to works; power of entry; obstructing persons in the performance of duties; recovery of fees, charges, and penalties; appeal; consolidated revenue; and the power to make regulations.

The regulations at present in force provide penalties for pollution, obstruction of or interference with the water in any stream or lake, and for causing trees, or débris of any kind to fall into any stream or lake, and also prescribes the various forms to be used under different sections of the Act.

From a perusal of these Acts,* it will be found that many of the principles advanced by the author have been provided for and are in successful operation in Australia.

The author certainly deserves the thanks of all water engineers for the exhaustive and thorough manner in which he has treated a most delicate and vexed subject.

Mr. Lewis.

JOHN H. LEWIS,† ASSOC. M. AM. SOC. C. E. (by letter).—Engineers are coming to realize the necessity for a more definite administration of streams by the public for the protection of existing and future acquired rights. They know what the law ought to be in order to accomplish the greatest good. Unfortunately, much confusion exists among lawyers as to just what the law really is, for any particular State, or for the Nation. Accordingly, a strong protest is made when any fundamental change is proposed. Little progress can be made in the reform of water laws, therefore, until such time as a large body of thinking citizens can come to some exact agreement as to just what legislation is needed, and then carry on a definite educational campaign to such end. Mr. Johnston's paper dealing with the fundamental principles of good water laws, therefore, should be a welcome addition to engineering literature.

It is pointed out that recognition of the doctrine of public ownership of property in water is the necessary foundation for public control, and that beneficial use should be the basis for establishing and maintaining rights to the use of public waters.

These definite principles are rapidly being accepted as the only logical basis for an administrative code of water laws. The necessity for the adoption of such laws is becoming more apparent each day, as the larger and more expensive projects for the utilization of water are taken up. It would be folly to invest millions of dollars in an undertaking unless firm legal rights to the necessary water can be secured in advance of construction. Further progress in the utilization of our streams demands that definite laws be enacted.

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* Copies of the Water Rights Acts at present in force in three of the Australian States, namely, Act No. 44, 1912, New South Wales; Act No. 2016, 1905, Victoria; and Act No. 25, 1910, Queensland; are on file in the Library of the Society.

† Salem, Ore.

The proper jurisdiction for the enforcement of such laws, however, is a matter of some uncertainty. Any discussion of fundamental principles, it seems to the writer, would be incomplete without some mention as to whether such laws should be administered by the County, the State, or the Nation, or perhaps by the drainage basin, irrespective of State lines. If we recognize the "public ownership of streams", then which of these political divisions shall act for the public in the administration of this public property? Mr. Lewis.

Agreement as to this point seems to be fundamental. In the days of small projects and abundant water, county control seemed adequate. Later, State control was found to be necessary. With the taking up of vast irrigation, drainage, water-power, flood-control, and navigation projects, involving the expenditure of millions of dollars and the building of structures in two or more States, it would seem necessary, in such cases, that the water laws be administered by some authority superior to that of the individual State.

At the outset, when the total investment was small, little attention was paid to the question of legal title to water; nor was much consideration given to problems of administration. If subsequent appropriators caused injury, the wrongful diversion was restrained by injunction proceedings in the Courts. The cost of litigation to preserve prior rights became in time so excessive as to compel early appropriators to insist on the adoption of modern administrative codes for their protection. These laws proved a boon to capital, as definite grants could be secured which would ripen into vested rights when the water was put to use.

Litigation between States over interstate waters is increasing. Eventually, diversions in some States must be enjoined for the protection of water users in lower States. The loss of time and money in protecting States' rights on interstate streams must eventually reach appalling proportions. Capital will hestitate to invest for fear the new diversion when made will be enjoined. The same difficulties which have been encountered between appropriators within a State will be encountered between appropriators in different States. State laws, however perfect, can give little relief. Stagnation in development and the great loss in time and money resulting from such an archaic system of "government by injunction" will compel the adoption of some Federal administrative code of water laws in the future.

The administration of streams is a public necessity. Both State and Nation have been derelict in their duty in this regard. Public rights have been shamefully trespassed upon in many cases for want of an administrative department with power to protect the public interest. Beneficial use is the proper basis for such administrative laws, whether State or National.

Mr. Lewis.

The unwise use of public property through lack of a definite administrative water policy for the greatest good to the greatest number occasionally results in great waste. This point may be illustrated by Snake River between Lewiston, Idaho, and Huntington, Ore., where the granting of a railway right of way may defeat the ultimate development of about 800 000 h.p. and the furnishing of water transportation to an empire greater than the combined areas of New York, New Hampshire, and Vermont.

Through lack of a public water department having full power to protect the public interests, we may wake up a few years hence and find that our public land department has granted land rights which will defeat public water rights of very great importance to the proper development of this region.

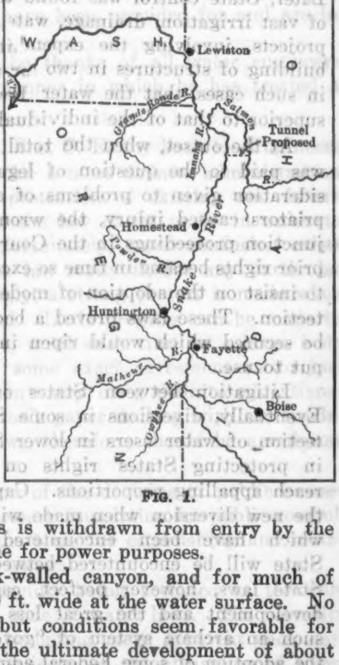

FIG. 1.

Between Lewiston and Huntington (Fig. 1) there is a fall of 1 300 ft. in Snake River, in a distance of 184 miles. The stream forms the boundary between Oregon and Idaho. At certain seasons it is navigable with great difficulty to Pittsburg Landing, 80 miles above Lewiston. This portion of the stream is doubtless under Federal control. Above this point, it is clearly non-navigable under present conditions, and is probably under the jurisdiction of the States with respect to water appropriations. One bank of this upper portion of the stream is in a National Forest, and the other bank for many miles is withdrawn from entry by the United States as being of great value for power purposes.

The river flows in a deep, rock-walled canyon, and for much of the distance is only from 300 to 400 ft. wide at the water surface. No foundation tests have been made, but conditions seem favorable for the construction of high dams and the ultimate development of about 800 000 h.p. Below the mouth of Salmon River, its low-water flow is approximately 10 000 sec-ft., and above that point 6 000 sec-ft.

On the profile, Fig. 2, there is sketched an ideal system of water-power dams arranged so as not to conflict with a 550-ft. tunnel project on Salmon River. This, with a 220-ft. dam and a 7- or 8-mile tunnel, would develop 200 000 h.p., with the 4 000 sec-ft. of water available at low stage.

Mr. Lewis.

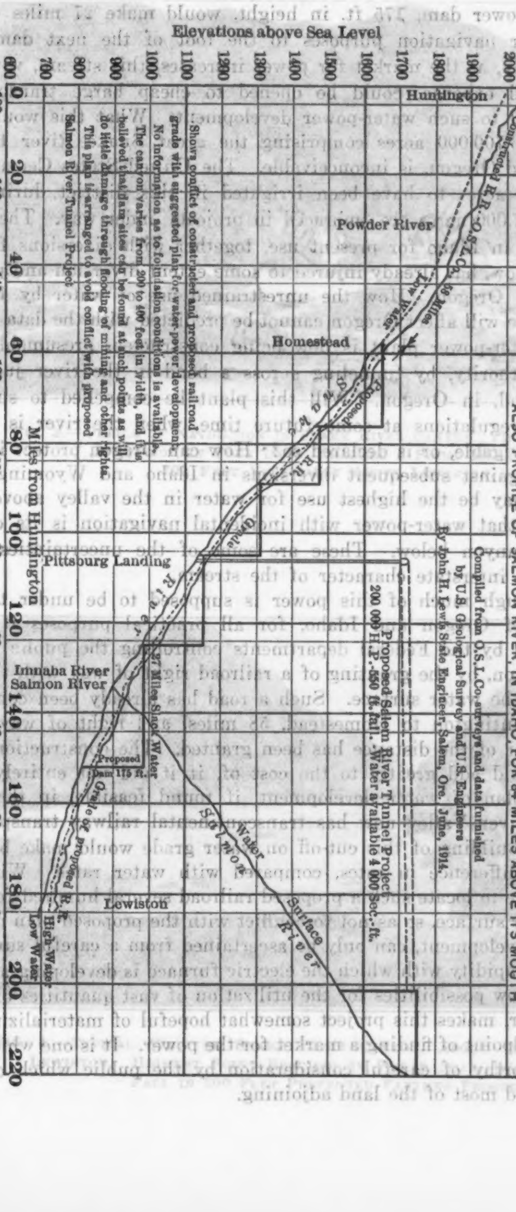

FIG. 2.

Mr. Lewis. The lower dam, 175 ft. in height, would make 27 miles of slack water for navigation purposes to the foot of the next dam above. Ultimately, as the market for power increases, this stream, with small additional expense, could be opened to cheap barge transportation incidental to such water-power development. What this would mean to the 43 700 000 acres comprising the great Snake River Basin in Idaho and Oregon, is inconceivable. The United States Census shows 1 611 000 acres to have been irrigated in this district during 1909, and 3 907 000 acres are embraced in projects under way. The storage of water in Idaho for present use, together with diversions from the regular flow, has already injured to some extent diversion and pumping rights in Oregon. How the unrestrained use of water by Idaho in the future will affect Oregon cannot be predicted with the data at hand.

A water-power plant is now being constructed, presumably under State authority, by tunneling across a bend in the river just above Homestead, in Oregon. Will this plant be compelled to submit to Federal regulations at some future time, when the river is actually made navigable, or is declared so? How can Oregon protect its water supply against subsequent diversions in Idaho and Wyoming? Irrigation may be the highest use for water in the valley above, yet it is clear that water-power with incidental navigation is its only use in the canyon below. These are some of the uncertainties arising from the interstate character of the stream.

Although much of this power is supposed to be under the joint control of Oregon and Idaho, for all practical purposes it can be destroyed by the Federal departments controlling the public lands in this canyon, by the granting of a railroad right of way with the grade close to the water surface. Such a road has already been constructed from Huntington to Homestead, 58 miles, and right of way for the remainder of the distance has been granted. The construction of this road would add greatly to the cost of, if it did not entirely defeat, such a plan of water development, if found feasible, in the future. Snake River Valley now has transcontinental railway transportation, and the building of this cut-off on water grade would make but little, if any, difference in rates, compared with water rates. Whether it is feasible to locate such a proposed railroad several hundred feet above the water surface, so as not to conflict with the proposed plan of water-power development, can only be ascertained from a careful survey.

The rapidity with which the electric furnace is developing and opening up new possibilities for the utilization of vast quantities of electrical power, makes this project somewhat hopeful of materializing, from the standpoint of finding a market for the power. It is one which seems to be worthy of careful consideration by the public which owns the water and most of the land adjoining.

FIG. 3.—PERILS OF NAVIGATION IN SNAKE RIVER CANYON. THE GASOLINE BOAT *Prospector*, IN FOUR DAYS, REACHED A POINT MORE THAN 700 FEET VERTICALLY ABOVE LEWISTON.

FIG. 4.—TYPICAL VIEW OF SNAKE RIVER CANYON, ABOUT 100 MILES ABOVE LEWISTON. HIGHEST POINT REACHED BY POWER BOAT, WHERE 9 FEET FALL IN 100 FEET PREVENTED FARTHER PROGRESS.

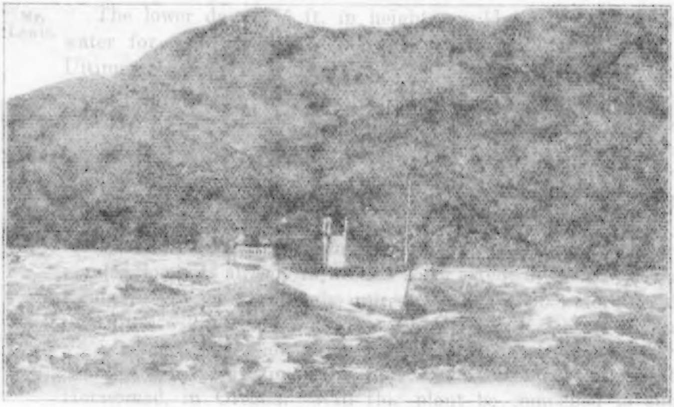

FIG. 3.—PERILS OF NAVIGATION IN SNAKE RIVER CANYON. THE GASOLINE BOAT STOPPED, IN FOUR DAYS, REACHED A POINT MORE THAN 700 FEET VERTICALLY ABOVE LEWISTON.

FIG. 4.—TYPICAL VIEW OF SNAKE RIVER CANYON, ABOUT 100 MILES ABOVE LEWISTON. HIGHEST POINT REACHED BY POWER BOAT, WHERE 9 FEET FALL IN 100 FEET PREVENTED FURTHER PROGRESS.

Mr. Lewis.

It is apparent in this case that the States cannot protect the public interest. The case of Snake River shows the confusion in the administration of related land and water resources among various departments which work no more in harmony than the various States. It shows the need of some new board with power to correlate the work of these various departments. It shows the waste which may come through unwise use of public property. It does not show the need of such a Federal administrative board for the protection of existing rights as clearly as some other examples, such as Bear River, which rises in Utah, flows through Wyoming and Idaho, and ends in Utah.

Many water problems are clearly beyond the power of the State to control. The United States has no administrative machinery to supplement that provided by the States. The problem is somewhat analogous to the regulation of railway rates.

What is apparently needed is an interstate water commission to handle all interstate, and international water matters, which are beyond the jurisdiction of the various States, just as we have an Interstate Commerce Commission to handle railway problems beyond the jurisdiction of State commissions. This water commission should have power to determine and record the various rights to water on interstate streams as serious controversies arise. It should have charge of the granting of water-power and other permits to appropriate water from all navigable and interstate streams beyond State jurisdiction. It should be charged with the distribution of water from such streams in accordance with recorded rights. In this way a reliable record of all future acquired rights would be available without the necessity of litigation.

Such a commission, with broad and general powers, could study the apparently conflicting laws and decisions of the various States and, in the course of time, through gradual changes, could ultimately bring about greater uniformity in State laws, and increase the effectiveness of the Federal laws. Congress cannot possibly give detailed consideration to each water permit to be granted in the future, and it seems to the writer that this work should be turned over to some commission or officer, whose duty it should be to handle all water matters so that eventually order may be brought out of the present chaotic condition relating to water titles.

Mr. Horton.

ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—This paper presents an argument in favor of the extension of the doctrine of priority in the use of streams, as now adopted, to some extent at least, in the Western United States and other irrigation countries, so as to include in this practice the eastern part of the United States, where the doctrine of riparian rights generally prevails. The writer feels

* Albany, N. Y.

Mr. Horton.

obliged to take exception to some of the author's arguments and conclusions; at the same time, it is recognized that there is vast need for improvement in riparian law and practice regarding water rights in the Eastern United States. How this may best be brought about is a problem yet to be solved. In one thing the writer agrees most heartily with Mr. Johnston, and that is that some system should be devised by which the tedious and expensive litigations now prevalent, both in the East and West, can largely be obviated. The writer also agrees with the author (and believes that most engineers in the East who have had wide experience in water litigation will also concur in the opinion) that many of the so-called judicial questions arising in water-right decisions could be settled far better by engineers, under certain judicial restrictions, than as they are at present by the Courts. The writer does not mean to cast any reflection on either the judicial or legal fraternity; perhaps the blame rests as much with the Engineering Profession as anywhere; but it is a fact that many water-right litigations resolve themselves, in simple language, into a war of experts, decided by a referee who is not competent, through knowledge, training, or experience, to judge exactly the merits of the contentions of either side. It has happened a number of times in the writer's experience that after a tedious and costly litigation over questions of water rights, a decision has been handed down by an entirely impartial judge or referee—a man of the highest ability from a legal standpoint—which was almost wholly incapable of practical application and required another lawsuit to decide the decision.

Looked at from an Easterner's viewpoint, it seems to the writer that the doctrine of priority, as applied to water rights, is fully as pernicious as the doctrine practiced in the East appears to the author. If the doctrine of priority is good, then the argument advanced by Mr. Johnston, that municipalities should be given the right to divert water from streams without compensation to vested power or other riparian interests, does not appear to be sound. The power interests are generally there first. If priority of use is a sound basis for establishment of right, then their rights are established thereby, and they should not be dispossessed without remuneration. The writer believes that in many instances vested interests have grown up in a manner inimical to public interest, and that curbing and regulating the use of public waters is necessary, but it must not be forgotten that the upbuilding of industrial welfare in the East is in a considerable degree the result of the utilization of water power by individual enterprise in early times, when land and water rights were unsalable or of little value.

The public has received, through taxes and other indirect benefits, enormous remuneration for the utilization of these rights, and, in the majority of instances, the writer fails to see where there is any

large element of unearned increment in value appertaining to the occupant or reputed owner of such rights. Mr. Horton.

Municipal governments are notoriously lacking in foresight in securing adequate sources of water supply. Suppose a city has grown to a population of 100 000 during the past century, has outgrown its existing sources of water supply, and must now take a new source from some large stream which, heretofore, has been developed extensively for power purposes. The writer cannot see any element of justice in a doctrine which would permit the municipality at this late date to seize without remuneration holdings on this stream, and divert the waters to its own uses. Had the city administration a century ago exercised the same foresight which was exercised by private enterprise in acquiring rights on the stream, then the present necessity for acquiring such rights after their intensive development by private enterprise would not exist. On the other hand, excessive awards undoubtedly have often been made in cases of municipal diversion from power owners. Legal barriers and obstructions have been placed in the way of a municipality seeking to secure better water supplies. To some extent public health has no doubt been jeopardized thereby for longer or shorter periods. This condition is primarily chargeable to the fact that all humanity is more or less human, and, being so, is greedy, grasping, and tenacious of its holdings; but, at the same time, it is believable that vast improvements could be made in methods of adjusting differences between municipalities seeking to divert water, and the owners of power interests affected thereby. The author's suggestion, that this can be accomplished through the utilization of engineers as arbitrators, with certain judicial functions, seems to be a good one. This method has not infrequently been tried in the East, and, so far as the writer has observed, the results have generally been more satisfactory, more promptly attained, and far less costly than when the usual condemnation proceedings have been followed.

The principal sources of litigation regarding water power, riparian rights, and diversion, in the East, as they have come under the writer's observation, are as follows:

1.—Land titles.

a.—Questions as to boundaries of grants on and along streams: whether they run to the bank or center of the stream, etc.;

b.—Questions of interpretation of terms used in land boundaries along streams, such as the "thread of the stream", "low-water mark", "ordinary stage", "high-water mark", etc.

2.—Flowage.

a.—Back-water from one dam affecting up-stream interests within the stream channel;

b.—Flooding of lands adjoining streams.

Mr. Horton.

3.—Diversion.

a.—By municipalities, as against water-power and riparian owners;

b.—As between owners on opposite banks of the stream;

c.—As between different owners on the same dam or power canal.

4.—Interpretation of grants.

Different classes of cases arising will be discussed later.

5.—Priorities and surplus rights.

6.—Prescription, abandonment, and unlicensed use.

7.—Transference of riparian rights, change in character of use, etc.

8.—Pondage and storage, or other interference with the natural regimen of the stream.

In the writer's opinion, litigations arising out of the interpretation of grants would not be mitigated in any way by the adoption at the present time of the doctrine of priorities.

The writer believes that existing grants, made years ago and indefinite in character, cannot now be reduced to simple terms, except by the tedious processes of litigation or by arbitration by a board of competent engineers. As illustrating the kind of questions arising from such grants, the usual language in which water was granted in the early days for power purposes at Eastern mill seats is briefly analyzed as follows:

1.—Enough Water for a Specific Purpose.—Grants of this kind have various degrees of indefiniteness, as, for example, quoting from the writer's experience: "Enough for a forge shop"; "enough for a saw-mill"; "enough for a woolen mill 20 ft. by 30 ft."; "enough to drive all the machinery which can be economically used in a cotton or woolen mill four stories high"; "enough for a fulling mill and oil mill, a carding mill, and enough to run a pump." The most common form of such grants was the use of the expression "enough water to drive one run of millstone." Grants of this kind were more or less indefinite. In some cases, the size and speed of the stone, kind of grain, and number of bushels to be ground per hour, were all stated. Such grants were capable of very definite interpretation. In many instances it is specified in such a grant that it conveys also enough water to supply power for the necessary machinery appurtenant thereto. In the majority of cases all that is stated is that the grant conveys enough water for a run of millstone, although the most common qualifications are the size of the stone, and whether or not power for attendant machinery is included.

2.—Enough Water for a Specified Water-Wheel.—Water rights in this language arise:

a.—From original grants;

b.—From prescription;

c.—From later interpretations of early indefinite grants.

Grants in terms of water enough for a specified water-wheel, if the wheel is a turbine of recent type and standard construction, are fairly capable of accurate interpretation. Unfortunately, however, many cases arise where the interpretation is difficult, as, for example, the grant of "enough water for a 40-in. Johnson re-acting turbine wheel as same was constructed in 1842." There were several types of Johnson wheels in use at about that time, and not all of them are the inventions of the same Johnson. Another grant which produced a litigation covering fifteen years, conveyed "enough water to drive three 6-ft. central discharge water-wheels." Grants conveying enough water for an overshot water-wheel 10 ft. in diameter, length not specified, and the like, are fairly common. Mr. Horton.

3.—Grants in Terms of Definite Fraction of the Flow of a Stream or Canal.—These are capable of interpretation if the capacity of the canal or the economical basis of power development in the stream are determined.

4.—Grants in Terms of a Certain Number of Square Inches.—In order to distinguish this term from miner's inches, it may be defined as a certain number of millwright's inches. This term was applied to the vent or capacity of water-wheels, and, when thus used, invariably meant a stream of water of the actual area specified, flowing with a theoretical velocity due to the head. However, when the term came to be applied to the definition of water rights, openings in flumes, etc., disputes arose as to whether it meant actual square inches of stream or square inches of opening, that is, whether or not a coefficient of contraction was to be applied. This question has been the source of many large and costly litigations.

5.—Enough Water to Fill a Certain Pipe.—Efforts to interpret grants of this kind illustrate the necessity for technical or hydraulic knowledge on the part of those intrusted with their elucidation. This was forcibly brought home, in one instance, by the engineer being asked by the Court how large a pipe a certain quantity of water would fill, after witness had stated the quantity of water to be diverted from a stream in terms of gallons per 24 hours.

6.—Grants in Terms of the Right to Take the Flow Through an Opening of Specific Dimensions.—Grants of this kind are productive of litigation as to form of opening, question of contraction, etc.

It is rare to find a water grant in the East in terms of cubic feet per second, or cubic feet per minute, or other definite language. As illustrating apparently the maximum degree of indefiniteness capable of being promulgated in a grant of land and water, the following approximate quotation is made from a deed of a water lot in Northern New York, a copy of which is in the writer's possession. The deed describes a parcel of land granted by bearings and distances, and ends

Mr. Horton. thus: "thence S. 28° W. 10 chs. 44 lks. to the southwest corner of the house now occupied by the widow Hawkins, be the same more or less." This deed was drawn by an old-time country surveyor and engineer, subject, apparently, to lapse of memory under certain trying circumstances, whereby he omitted to specify the acreage in this case.

As a matter of curiosity, the writer compiled the principal causes involved in some fifty water cases within his experience. The results are as follows: back-water, 8 cases; interpretation of grants, 8 cases; diversion, 14 cases; appropriation, 17 cases; and other causes, 3 cases.

Diversion and appropriation should properly be classed together; they comprise 60% of the total. This suggests forcibly the necessity of better methods in handling diversion cases than those now prevailing.

Mr. Decker. GEORGE P. DECKER,* ESQ. (by letter).—The civil engineer has addressed himself in earnest to the water law problem. There are signs of it on all sides. Lawyer and layman must welcome the fact. The uses to which surface waters in natural bodies are put, has distinguished water from upland radically in connection with private rights. Water may be the subject of certain exclusive individual uses, but not more so than air. The uses of water, in most cases, are enjoyed in common by all members of the community, as are the uses of the air. Want of fixity of water distinguishes its character from that of land. The latter may be subdivided and the subdivisions may be enjoyed exclusively and harmoniously by individuals, but the former may not. These differences have resulted generally in recognition, sooner or later, of a necessity for governmental administration over water uses.

In newly settled countries, such as our States were recently, this administration begins with simple rules, and these increase in complexity as need for all the various water uses increases. Differences in natural surroundings lead to a different order in the development of such rules. In arid regions rules applicable to irrigation may be first developed, or may be the only ones developed. Along tidal waters rules are needed as to the use of riparian frontage in commerce much more than on interior waters. On power streams rules of a different sort, when developed at all, are found, and these are adjusted to those for other uses, according to the order or extent of the development of local industries. The common right of fishing is world-old; yet we have had State judges who have denied that it existed as to fresh waters. The denial of that common right was quite necessary in the effort to sustain the argument for denying the public right to water-power uses. Rules for the regulation of navigation, a common right, were developed at an early date, and so was active governmental admin-

* Rochester, N. Y.

istration over that right, because, in early days, of public dependence on streams for travel; the small stream as well as the large one. At first, the private user of water power was an outlaw; the power use might be enjoyed only where, and so long as, it did not interfere with boating—then the more important use. The power use, however, was soon legalized in all the States by exp... statutes, in order to protect the private user from prosecution in the name of the public, for his use was not at all in common with that of other members of the community. His use was exclusive, and, because of the necessary interference with the stream, was hostile to community uses for boating and also for fishing. The legalizing of that use by him was merely a temporary permission, a franchise, vouchsafed by the public. As long, however, as there was power enough, generally speaking, for every one, anything like real governmental supervision over it was in abeyance. In the United States that condition has lasted from the beginning almost to date, as to all active governmental supervision over public uses, municipal uses so-called and navigation being general exceptions, and irrigation a special exception in arid States. Mr. Decker.

Absence of supervision invited wide-spread litigation between conflicting private interests, and from that grew up a body of judge-made laws as to water uses, which came to constitute not only our water law so-called, but the law as recognized and widely applied. Mr. Johnston's criticisms of the quality of that law are admirably put and wholly justified. The only water uses in the Eastern States, which have demanded governmental supervision, so that these uses might be equitably enjoyed, were those of navigation and for municipal purposes. In applying that administration, however, the public was confronted with strange doctrines developed in the judicial decisions which had accumulated. It found that the exclusive use of a stream for power, enjoyed by the miller, had been recognized by the Courts as having a foundation in private property, as a vested right, not only as against his private neighbor, but as against all the rest of the public represented in its sovereignty by Government. That doctrine has been repudiated very recently by the United States Supreme Court in the Chandler-Dunbar Company case, arising at Sault Ste. Marie, and at intervals in earlier cases, some of them in the Courts of various States.

The enormous population with which we are now confronted, presenting itself almost everywhere, makes its own demand for a common enjoyment of the naturally public uses of water and for the enforcement of the public right thereto. We face a necessity, therefore, for active governmental administration of all such uses, for without it any real public enjoyment is quite out of the question. The rights of the public, of course, will be respected. That they should be disregarded for long, after enjoyment of them has become essential to public

Mr. Decker. welfare, is impossible to conceive. No judge can rule irrepealably that these uses shall, for all time, at this place or at that, constitute private property in the few owners of adjoining upland. A new judge can be chosen who will overrule him. The development of the policies to be followed and the methods of administration of the various uses to which the public is entitled in bringing the common right or benefit to all, are among the most important tasks confronting the American people. The Engineering Profession, of course, must play its part. It is, in fact, the only profession qualified to-day to undertake that task. The department of the law which the legal profession has most ignored is that which deals with waters. No student entering that profession, up to the present, has had facilities for specializing in the law of waters, if he had desired. In the course offered by law schools, the law of waters is not usually dignified with provision for special instruction. Few practitioners have been in such circumstances that they could devote themselves specially to this subject. The water cases which have come before the Courts have been taken there by lawyers who in almost every instance were engaged in their first and last water case. The judge who decided a water case and who was sure to write an opinion, possibly—if not probably—was deciding his first and last water case. The result has been that there is no group of decisions so devoid of uniformity in the application of principles, so contradictory in the rules applied, as those which have gone to make up our water laws. No State has yet developed an adequate code of water laws. Such provisions as exist are usually found scattered through statute laws on other subjects, partly in the highway law, partly in the fish and game law, and partly in the municipal law. Only a civil engineer could have enjoyed a training and experience which would lead to the broad view of the subject revealed by Mr. Johnston's paper.

Mr. Johnston declares, as a first principle, the "public ownership of streams, lakes, and other bodies of surface water." That statement is useful at this time for the purpose of pointing out the common and great error involved in the statement that there is a qualified or almost unqualified private ownership. The use of the term "ownership" in connection with the subject of waters is always unfortunate, for it leads to misconceptions as to rights. It has been the layman, however, who was responsible for introducing this term in this connection, and he demands perhaps that it be used.

It is desirable in the early stages of the effort to introduce a sane and just system of water laws, in order to eliminate all error possible to be implied in the statement of principles. The writer is of the opinion that Mr. Johnston's statement embraces such an element. Although water uses include many which are naturally to be enjoyed in common by all members of the community, there is one at least which is not

naturally shared in this way under our system of private ownership of upland. The private owner of upland adjoining a stream is, quite independently of any act of his or of interference with a stream by him, served with those benefits which come through the saturation of his land by the stream. Such a benefit may justly be termed a natural one. It is one which the earliest agricultural settler relied on in securing his location. On that saturation, certain crops may depend. Riparian lots available for dwelling purposes may also have a value, due to the natural beauty of the contiguous stream. If so, that value was placed there by Nature. The stream may have a flow sufficient to be serviceable as a highway for boating, and, if it has, it was rendered so (opened) by Nature and not, as a rule, by Government, and is an outlet for the riparian land owner. In the case of highway openings by Government and the consequent invitation to settle and improve locations adjoining it, on the assumption that the highway will be perpetuated, it is recognized as just, if not indispensable, in closing the highway, or in shifting it, or in making successive changes in street grade, that compensation should be paid the abutting owner according to the injury which may result. Mr. Decker.

Diversion of a stream, then, without recognizing any obligation of the public to pay any damages—diversion on the theory that the public owns the stream (may destroy it to the extent of diversion) and render no compensation to any one—may involve injustice. This view is not based on recognition of any right in a private abutting land owner to interfere artificially with a stream, to divert, pollute, or obstruct it. The benefits referred to are such as are natural. If stream flow is to be diverted, then, by the public, in connection with an intended improvement facilitating public enjoyment of a common use or uses, or if a fraction of the flow is to be diverted, just damages according to the injury done should be paid. Such cases would be unlike that of the removal of a public building, artificial in itself, and where the gratuitous benefits which had come to neighboring lands by its construction would be lost on its removal. If taken away without compensation, there has been no injustice inflicted by the public. The law as administered has usually failed in the proper grouping of the cases which may afford a just ground for private damages against the public in case of complete or partial diversion of stream flow. If, however, damages are to be awarded when justly called for, as here indicated, no serious obstacle in the way of the proper development of streams for their highest public usefulness would be presented.

The deterring of public improvements and of desirable governmental supervision over streams, has been due to the gross injustice to the public arising so often when damages have been awarded on the mistaken theory that uses which were in principle public and common

Mr. Decker. were exclusively private, and private in perpetuity. The commonest example of that has been in case of power uses, for those, when treated as based on rights of private property, exclusive and perpetual, and then capitalized on the basis of their economic value, lead to startling figures in the making up of Court awards. If a private property right, as against the public, to use the water power of a stream for private purposes when the public needs it for lockages, cannot be sustained in justice, the same reasoning should apply where the public needs to divert the stream for municipal supply or for public power purposes. The stream in any such case would be taken for public purposes. It is for the Government, through its legislative department, to say which of all public uses shall have the right of way over the others. The riparian owner, from whose private frontage a stream is diverted, then should be paid no element of damages on the theory that as such land owner he has any exclusive right of use of the stream at his front for boating, for fishing, for drinking, or for artificial diversion for ends of his own. His damages in case of diversion by the public, when confined to loss of natural benefits, would consist of a nominal award in most cases, and a modest award in any case—so small in most cases that he would not seek an award.

A statement of principles based on recognition of the public right may be formulated as follows: Water uses which the public may enjoy in common are inalienable by Government for private benefit; they are subject at all times to regulation and administration by Government; if the public diverts a stream, it is liable for the injury, if any, produced by cutting off the natural irrigation or other natural benefits received from the stream by private riparian lands; if the channel below the diversion was navigable, the public is also liable for the consequent depreciation, if any, in the value of the lands fronting thereon; if the public is to acquire the right to divert forever, the damages, of course, will be estimated accordingly.

We seem already to have reached the time when denial, tantamount to betrayal, by State Courts of the public rights in respect to water uses must cease. Two regrettable instances of that have occurred recently, one in the Court of Appeals of New York, in the Fulton Light, Heat and Power Company case, the other in the Supreme Court of Wisconsin, in the Wausau Street Railroad Company case. In both these cases, the Courts applied the principle requiring protection of private property provided for in Federal and State Constitutions to safeguard it from unjust attacks by the public. It is now necessary that all concerned shall recognize the principle of greater sanctity and importance, namely, that public rights shall be safeguarded against attack by private interests. This principle is so fundamental the world over that constitution makers have not deemed it necessary to affirm it specially. It applies in the case of very many fundamental

rights, of which no mention is made in written constitutions. It is gratifying to know that the highest judicial tribunal in the United States respected and enforced this principle in the case of a water use subject to public enjoyment, in the decision reached in the Sault Ste. Marie case in 1913 (229 U. S., 53), where a lower Court award of $500 000 against the public, in favor of a private corporation claiming a monopoly in the use of that river for private enjoyment, was reversed. Mr. Decker.

Clarence T. Johnston,* M. Am. Soc. C. E. (by letter).—When we view land, there are two principal features of topography which impress us. The first is area, or the horizontal projection of the surface of the ground. The engineer measures this; the Government grants title to it; and private parties buy and sell it. Then there is elevation, which gives life and profile to landscape. This may be called the vertical projection of the ground surface. The first feature is stable, there is only one horizontal projection. The second feature depends on the location of the observer. Every time he changes his position, the scene changes. If altitude did not exist, there would be no scenery, or if there were, it would be monotonous. People who live in a flat country always lack some sources of inspiration. The vertical projection of land is of special public interest. The public does not part with this when title is given to horizontal areas. It cannot be bought and sold, because it supports art, beauty, sentiment, and fundamental principles. Private parties may change the surface of the ground in detail, shifting the vertical elements slightly, and still not transgress on the rights of those who appreciate and enjoy gifts which naturally belong to all. Mr. Johnston.

It is because of the vertical component in landscapes that streams flow and lakes exist. Lakes form because a contour—a line of equal elevation—is continuous on the surface of the ground; the lands within the boundary of this contour being lower, water collects. Streams flow because the ground over which the water runs, has a constantly decreasing elevation, from the source of the river to the sea. As a stream is followed toward its source, it breaks up into innumerable tributaries, and these continue to sub-divide until the channels are almost indiscernible. This stream, its tributaries, and the many small channels, constitute a natural drainage system. Where a country is blessed with ample slope, that is, where the second feature of topography is represented in a valuable way, drainage is generally natural, and Man is relieved of much labor and responsibility. Where this feature of topography is wanting, in some degree, he must do more for himself. Streams and lakes, therefore, are of public character by decree of Nature. The vertical element of topography prevails throughout the drainage basins of streams, and it is present in and along their chan-

* Ann Arbor, Mich.

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nels. Where rapids or falls occur, this component is manifested in a striking way. Without analyzing the principles on which the action has been taken, the public has preserved waterfalls, in some measure, for public benefit. The vertical element of topography in the channels of streams has an influence on irrigation, navigation, and the development of power, all, shall we say, matters of public concern. As the land to the right and left of the channel of the stream is investigated, the vertical element of the topography becomes more prominent and striking. There is nothing in Nature to indicate that an owner of riparian lands is more interested in the stream—except in so far as proximity is concerned—than is the owner of lands more remote from the river. The river serves both in matters of drainage, navigation, and possibly irrigation. It should furnish both with power and with water for domestic purposes, in so far as these may be needed. Nature, evidently, did not suggest the doctrine of riparian rights to Man.

When the Courts apply the common-law doctrine to streams and lakes, they go beyond legislative halls and the schools of Nature to secure the rules which govern. The doctrine of riparian rights could not live in the atmospheres of modern Governments were it not changed frequently in efforts to meet new conditions. The Court assumed legislative authority, in a limited way, when the doctrine was introduced, and each time new constructions are made, the judiciary invades the field of the lawmaker. When a Court finds the construction which it approves, a decree is issued; then another difficulty is presented. Because legislation, produced in this way, is not framed on broad lines, no administration is available to carry the rules of the Court into effect. The Courts have not hesitated when confronted by a serious obstacle of this kind. They appoint Court commissioners, or other officers, and take responsible charge of the administration. Since the Constitution was ratified, therefore, the judicial branch of the Government has been able to maintain its own exclusive field and to grasp something in both the legislative and executive departments.

The definition of the doctrine of riparian rights, given by Mr. Bellamy (p. 649), is almost identical with that quoted by the writer (p. 634). The case, Mason *v.* Hill,* decided in 1833, records an epoch in the development of the doctrine. Prior to this decision there seems to have been some tendency, even in England, to give weight to priority. It breaks down any further claim in that direction. As early as 1805 Lord Ellenborough said:

"The general rule of law as applied to this subject is that, independent of any particular enjoyment used to be had by another, every man has a right to have the advantage of a flow of water in his own land without diminution or alteration."

* 5 Barn. and Adol. 1, 110 Eng. Reprint, 692.

The most notorious case in the United States, where the riparian doctrine has been attacked, is that of Lux *v.* Haggin.* In this case, the conflicting claims of riparian owners and irrigators were brought to the attention of the Court. The decision was favorable to the riparian proprietors in the Lower Court, and an appeal was taken to the Supreme Court of the State. In the latter Court seven judges solemnly listened to testimony which revealed a war between the ancient and the modern, the impractical and the practical, the common law and statutory provisions pointing to something better. Four of these judges upheld the decision of the Lower Court, the principal point being that the Court had no power to modify the common law. As the common-law doctrine had been manufactured by Courts and repeatedly construed and misconstrued by them, one can with difficulty understand their caution on this occasion. The dissenting opinion, written by Judge Ross, is an able document, considering that it was prepared thirty years ago when little was known, in the United States, of principles relating to the administration of streams. He defines the riparian doctrine as follows: Mr. Johnston.

"By the common law, the water flowing in a river or stream, or down a cañon or ravine, could not be acquired by appropriation, and must continue to flow in its natural channel undiminished in quantity and unaffected in quality."

The judges who wrote the prevailing opinion quote several definitions which are almost identical in meaning. Although the case resulted in a victory for the riparian claimants, the Court did not attempt to determine the rights of the interested riparian proprietors.

If the riparian rights were limited to those which the owners of riparian lands naturally enjoy, very little trouble would follow its application. The owner of riparian land has advantages because of the proximity of the stream. It furnishes him a convenient source of water supply for all domestic purposes. It may be of value to him in the way of transportation. Domestic uses imply diversions from streams, yet these exist even in countries where the doctrine of riparian rights is presumed to be observed. A doctrine which permits (in practice rather than in theory) of a small diversion for a use which supports life, must permit of large diversions for the same use, or the doctrine fails. A doctrine which compels a town to pay tribute to an owner of riparian lands, regardless of beneficial use or preferred use, does not merit praise or support. Because the owner of riparian lands has natural advantages, other favors, of more weight and importance, should not be extended to him. The natural uses of the owner of riparian lands should be protected, but he should not be permitted to impose on others whose needs are as crying as his.

* 69 Cal., 255, at 398 *et seq.*, 10 Pac., 674.

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If engineers throughout the United States realized the problems that are presented in the administration of streams, more rapid progress would be made in the enactment of wise laws relating thereto. There are engineers who submit without question when they are told that this or that is the rule, and they are not inclined to study conditions elsewhere, apparently fearing that they might ultimately be led to recommend innovations which might not be appreciated, in the beginning at least, by employers. The engineer who specializes as an expert witness seldom progresses very far in the study of principles. Principles cannot be bought in the open market, and a discussion of principles by a witness is seldom countenanced. There are also engineers who constantly look forward to the creation of offices which may demand engineering talent, and any suggestion that principles be studied immediately reminds them that some officer must carry these principles into effect. Their usefulness in the great field of initiative is ruined, because they prefer to discuss machinery rather than purpose and design.

It is important, it will be admitted, whether or not an administration is to be under the control of National, State, county, or township officers. As water users, we would not worry greatly, regardless of the character of the administration, provided the principles, so necessary for a just division and distribution of the available water supply, were carried into effect without fear or favor. Further, after we have discovered all the principles that have been approved by the best authorities, we may be able to say what kind of an administration we need. It is possible that some or all of these principles should be incorporated in a National statute and be administered by National officers, as Mr. Lewis recommends; some may properly go to the States; others to the counties, and still others to lesser subdivisions of government, or to drainage basins, regardless of political lines. All kinds of interstate trouble may be imagined. We can go further and discover international problems. Thus far, we have had a little trouble on the Rio Grande and on the St. Lawrence. It might be best to have all streams under international control. However, we can well spend our time studying principles and, later, determine the kind of an administration we believe is necessary to carry them into effect. To have the administration first and an enunciation of principles later might lead some to believe that the demand for office exceeds respect for principle. Generally, under these conditions, the principles which should govern are not separated from rules, which have no logical application, until long after the administration has made its way, limpingly and painfully, along a road that does not run parallel with efficient public service. It would be much better to have every State become interested in a discussion of principles than to provide interstate commissions to regulate the use of water from streams.

To establish these commissions now would simply mean that a deadlock would follow their first serious attempt to do business. Colorado, the procedure of which is based on principles not too plainly defined by law, is on a different basis from Kansas, which relies on the doctrine of riparian rights. Commissions made up of men from the two States would have to go back many centuries to secure common ground, and then they would find themselves unsupported by the laws or customs of either Commonwealth. A National commission might have an experience almost as discouraging. Mr. Johnston.

Although all these troubles, and many others, would arise, and although a commission of engineers would have a rough way to travel, until wise principles were found and incorporated in the law, yet such a commission would be able to record some progress each day. Engineers understand the natural laws which govern streams. They know how the laws of Man must be framed in order that these natural laws be not violated. Bearing these things in mind, they are in position to study principles on which the man-made law should be based; so that the commission, wholly ignorant in the beginning of the principles which should govern, is to be preferred to the Court. The commission makes progress. The Court does not. However, the commission may fail while it is searching for principles. If, within a reasonable time, the commission cannot announce some of the great principles for which it should stand, the law creating it may be repealed. The study of principles, therefore, should precede the creation of office.

The principle of priority is dangerous when it stands alone. When the doctrine of riparian rights was abrogated in some of the western States, the principle of priority was introduced, largely by the Courts, and it was unsupported by other principles for a long time. The Courts never discovered the principles that must go with and supplement the principle of priority in order to produce a thoroughly balanced administration. As soon as the principle of priority replaced the common-law rule, the Courts began to build on it a network of theory which brought almost as much injustice as was suffered under the original doctrine. Those whose business instincts had been sharpened by a somewhat rough experience, laid plans to seize streams under the protection of the new principle and by the consent of the administrators, the Courts. All kinds of testimony were brought to the Court. Though the Court kept the principle of priority in view, it did not know how to determine the dates which fixed the priority of rights. Time might begin to run when the prospective water user first thought of using the water, when he began a survey in the field, when he began or when he finished construction. The Courts did not know how to establish the limits of the rights to use water. In some cases, the right was limited by the claims made by the applicant in some office of public record; in others, by the size of the

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conduit which carried the water to the place where it was to be used. The Courts never made much progress when it came to limiting the right by the actual demands of the use, but were inclined to divide water among claimants and then view the rights thus defined as private property which might be bought and sold. These are only a few of the complications that arose, and yet the principle of priority was still observed. It was not supplemented by the few other principles that have already been briefly described. The complications which Mr. Horton describes, and his criticism of the principle of priority, as he sees it, are to be anticipated. He has enumerated some of the things which give rise to litigation. These will be referred to by number and letter as he has designated them.

1*a*.—The determination of boundaries of "grants" has become necessary, it must be supposed, because a few people, presuming to represent the public, have assumed to give to private parties a title in and to the beds of streams and perhaps to the streams themselves. If all uses are to be protected, this kind of property cannot be taken from the public, and if there is a title resting anywhere, it at all times lies with the public. Even Courts, sooner or later, must come to this conclusion. If all users understood that the stream itself, including the channel and the water, is always public, there would be little occasion for a contest as to the location of such boundary lines as are generally concerned in litigation of this kind.

1*b*.—The difficulties which arise under this heading can be ascribed largely to the troubles which always prevail when private parties are given "rights" which concern, in any way, the "thread of the stream", "low-water mark", or "ordinary stage." The "high-water mark" should define the boundary between private and public property, and this should be located by the public, once and for all time. This at once assumes an administration. The administration is necessary to see to it that public interests are preserved and that private uses do not conflict. Under these conditions, the public, only, would be interested in the "thread of the stream", "low-water mark", and "ordinary stage", and, in a few years, we would never hear the terms except when reviewing literature relating to public property and political boundaries.

2*a*.—The problems presented by "flowage" are also simple when an administration is provided. With a central office of record and a knowledge of each stream, with laws requiring prospective water users to file applications and complete plans, all conflicts between power plants and other uses may be provided against before any injurious use is initiated.

2*b*.—When plans are submitted to the office of record, the public at once knows what lands must be flooded, and the parties proposing to bring about development are then obliged to deal with the land

owners. Where public interests are involved, even in works proposed to be undertaken by private parties, a simple and direct method of condemnation should be provided. Should the parties not be able to proceed with construction within a reasonable time after these difficulties are removed, the permit issued to them should be canceled. Mr. Johnston.

3a.—The diversion of water by cities, to the detriment of water-power plants, is now and will continue to be a source of litigation until a better understanding is reached. Mr. Horton is right when he says that cities have been "notoriously lacking in foresight in securing adequate sources of water supply." This is true, because the city represents the public, locally, at least. Our customs, if not our laws, generally favor the individual and the corporation as against the municipality. The city has been imposed on whenever the common-law doctrine has been invoked. Those doing business for the city have not had the best opportunity to gain the experience which would tend to develop and support an able administration. Every city has a small beginning. It is then in keen competition with private individuals. As some of these individuals are likely to be in partial charge of the municipal government, the city administration does not start with the best of prospects. Before the town has cast aside its swaddling clothes, the streets are largely occupied by things that belong to outsiders. The city fathers have one duty which appeals to them pre-eminently—the granting of franchises. Almost everything that concerns the convenience and the happiness of the people is given over to the management of private corporations, with the single and glaring exception of the sewer system. This is likely to be poor in design and construction, and to promise no profit in operation. It is, evidently, a good thing for the city to run, and future city fathers may obtain their best experience as administrators therein and thereby. It is not surprising that cities are "notoriously lacking in foresight in securing adequate sources of water supply."

3b.—The troubles which may arise between users who divert water on opposite sides of a stream are probably numerous where an attempt is made to administer rivers and lakes under a doctrine which, in its fundamental form, makes diversions impossible. If the relative rights of each were to be defined by public authority, and each was required to live up to the terms of a specific agreement on record in the central administrative office, we would not anticipate much uncertainty as to the limits of rights and uses, and consequently we would not look for much, if any, litigation. The rule of priority, properly supported by other important principles, might be found to offer salvation under such conditions.

3c.—If the individual owners of a power plant, dam, or canal are inclined to quarrel, they should have permission to do so, as long as they conduct it as a family affair and make no attempt to stir up

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strife among those who have their rights defined and who, consequently, understand their relation to each other and to the public. It has been demonstrated, however, that where an administrative control is exercised, there is little cause for uncertainty, and uncertainties always lead to trouble. Often those interested in an individual plant quarrel relative to the title of property, which, of right, they do not own or control in any way. Usually, the property concerned belongs to the public. As soon as the public assumes control of that which it naturally owns, a great temptation is removed.

4.—The interpretation of grants generally means that the individual has assumed the ownership of public property and has doubts as to his ability to perfect the title. Grants are iniquitous. They mislead the public and the individual. They can develop under any system which is careless. Claimants have demanded as much as 500 cu. ft. of water per sec., and to be delivered through a pipe 1 in. in diameter. Another called for 60 cu. ft. per sec. and proposed to convey it in a ditch 15 in. wide and 10 in. deep. These suggest problems for expert witnesses in litigation which generally leads to prodigal decisions. It is much easier for a Court to prepare a decree, relating to matters foreign to the law, than it is to carry such a decree into effect on the ground. Grants by other, unauthorized, public authorities are generally of equally questionable value.

These grants must all be modified in time. Some day the user will meet a public officer who operates under a law based on wise principles. He will give his testimony and be afforded an opportunity to scrutinize the testimony submitted by his neighbors. A hearing will then be provided for, and at this gathering of all interested claimants, adjustments will be made which will satisfy all, that each is to be protected, that there is to be no favoritism, and that public interests are to be preserved. Finally, an administrative board, respecting all the great and important principles which protect water users and uphold public interests, will prepare a "decree" defining the various rights in a way that all may understand. Then an engineer of average ability will carry the order into effect. The determination of rights will be made without expense to the water users. Throughout this procedure all claimants will have the same opportunity to be heard and have their interests protected, regardless of financial ability. No litigation will ensue. Water users will be satisfied.

5.—As stated previously, the principle of priority does not bring order when it stands alone. When it is supported by other principles, some of which have been enumerated, "surplus rights" do not exist. Injustice always results where water users are permitted to expand their own uses or to sell "surplus water" to others. In the early days of the West, when the principle of priority stood almost alone, the Courts were inclined to give excessive allotments of water. In one

case, a prospective user filed a claim to nearly 1 000 cu. ft. per sec. from a stream that did not carry that much during the summer months. He used only a small part of this volume, but 16 years later, after many other uses had developed, he was permitted to sell what he did not need for his own purposes. The granting of "surplus rights" always results in litigation. It is unfortunate that those injured are generally obliged to resort to the same tribunal for relief as that responsible for the existing condition. It is natural, under these conditions, that relief comes slowly. Mr. Johnston.

6.—Prescription is an outgrowth of the doctrine of riparian rights. This doctrine does not permit of the diversion of water from streams. Where the Courts find that it has been violated for a number of years and money has been spent in construction, the water user is protected under the theory of prescription. Prescription is the reward extended to water users who violate the doctrine of riparian rights. It is natural that it should lead to some trouble. It was born of trouble. It is almost as iniquitous as the doctrine of riparian rights. Regarding abandonment, the public should insist on the beneficial use of water. If use is not made, there should be a way to terminate the right. The period of non-use should be brief. With so many concerned, and because waters are of so much value to individuals and to the public, the latter cannot consent to witness even a brief period of non-use. The reward which the public exacts at present is less than nothing. To demand continued, beneficial use, does not place the public on a par with the private consumer. The Courts are very slow even to call the attention of private users to the fact that they are slothful. "Unlicensed use" is a new complication, and the terms would suggest a suspicion of public interest somewhere.

7.—The transfer of riparian rights, or almost any right or use to another use, is generally fruitful of litigation. That is why the right to use should belong to the use and not to the user, and always be limited to the quantity applied beneficially. There has probably been no more frequent or serious injustice done water users than that which has resulted from transfers of this kind. A riparian proprietor generally uses water for domestic purposes. Instead of calling him a riparian proprietor, or a proprietor of water or water rights of any kind or degree, he should be known as a water user, the kind of use should be defined, and a limit placed thereon. He is blessed by Nature because he has a supply of water at hand. Under any competent administration he has nothing to sell in the way of water or water rights. If others buy his land, they probably pay more, slightly more, because of the proximity of water supply. If some one desires to flood his land, or a part of it, in order that a pond may be created, he sells a part or all of his holdings. He does not sell water or water rights, because he may still wish to exercise his right to domestic use, and,

Mr. Johnston. if not, that right lapses. It is the only water right he has. The public still owns the water and the channel of the stream. When the land is flooded, no change occurs except that the purchaser of the land takes the place of the original occupant, and there is a water expanse instead of a ground area. The channel of the stream and the water still remain public. Under the doctrine of riparian rights, the stream cannot be dammed or the flow of water interfered with. That doctrine survives, only to be violated.

Change in character of use should always mean forfeiture of rights, except where the use is to be transferred to a preferred use. It is practically a forfeiture in the latter case, also, except where the principle of priority is upheld. Under the latter condition, the priority is carried with the transferred right to use water. The use can always be disposed of, and the public records do not need to be burdened with the names of users. The public records, in so far as the administration of streams is concerned, simply relate to the uses, their priority, their location, and their extent. These elements are not altered by any change in the ownership of property connected with the use.

8.—The natural flow of streams may be affected by pondage, storage, and by actual diversion. All these things are prohibited by a doctrine which gives every owner of lands abutting on the streams the right to demand the waters of that stream to pass his lands undiminished in quantity and undefiled in quality. Yet these things all prevail, on every stream where the doctrine of riparian rights is presumed to be the mainspring of inspiration, in the United States. Pondage, storage, and diversions would not result in litigation, or even serious conflict, under an administration supported by the principles described.

Though public ownership and supervision of water is of great importance to the public, whether represented as a Nation, a State, or a lesser political subdivision, it is of greater value and importance to water users, who have led in the demand for public control of streams and lakes. Where water users oppose legislation of this kind, we must come to one of possibly two conclusions: Either the public concerned lacks confidence in itself, or water users are ignorant of what has been accomplished where public ownership and supervision have been tried.

Mr. Decker makes a very interesting presentation of the relative rights of public and private users. Where the public diverts water from a stream to the detriment of other users, some return from the public, in proportion to the damage done, is natural and just. However, in making this transfer from one use to another, it should be borne in mind that, as the public use grows, the private use decreases, and that the public is not buying water rights or water, but is paying damages to the private users because of the decreasing efficiency of the plant concerned in the use, due to partial failure of the water supply. The local public is then taking water from a river that be-

longs and always has belonged to the public generally. This water has merely been used for a brief period by the private parties. They have invested money and built a plant or plants for utilizing the water in some way. It is not a great undertaking to determine what damage is done when the water supply is reduced by a known volume. The city is generally required to pay for water rights, and other intangible things, on a valuation which would never be countenanced were the private party arranging to reimburse the city in a similar transaction. Assume, for instance, that a city is using the entire flow of the stream for power purposes. A new use then develops. This new use, we will assume, is created by the needs of a farmer and his family. The farmer uses a small volume of water daily, and he must divert this above the power plant of the city. A user of this kind is generally protected, provided he can have access to the river. If he is an owner of riparian lands, the Courts uphold him to the extent of his domestic uses, at least. How the Courts protect the use under the common-law doctrine, we need not try to understand. As we assume a larger number of farmers, each making an equal demand on the river, the city must gradually relinquish its use of water for power purposes. Now, the owner of riparian lands, under the common-law rule (so the Courts tell us), can use the water for domestic purposes and refuse to reimburse the city. The principle of priority would have to be recognized, we must assume in such a case, before the city could demand a return for damage done. Usually, we have the domestic uses of a city against the power interests of private parties. The domestic uses of a city support life. If the private user for power purposes has his money returned to him, so that he loses nothing by the investment in power development, society at large receives the highest, general benefit. By assuming that the public, the municipality, is the user of water for power purposes, and that it must give up that use as more important and necessary uses develop, the question may be viewed in a light that is possibly new to many who have studied it from other angles. Mr. Johnston.

An example from an adjoining field may serve as an illustration. A city is asked to extend a franchise. Representing the public, it is naturally presumed to give the franchise to some company which flatters itself with the announcement that it is to provide a "public utility." The franchise is given, the only condition attached being the construction of certain works, a part of the "public utility." Before the franchise expires, the city decides to purchase the private, corporate property and try to do something for itself. We might assume that the corporation would volunteer to turn over the property at actual cost, less depreciation, plus appreciation, plus renewals and replacements, etc., etc. The city is generally charged a tidy sum for such part of the franchise as has not expired. If the city had considered the value of the right to do business in the beginning, it could, by

Mr. Johnston. no possible argument, have been shown that a franchise should ever have been given, because it had the opportunity then to serve itself in perpetuity.

An example within an example may not be out of place. A new road is to be opened. Some private lands must be secured for highway purposes. The public has power to condemn lands for such purposes. The value of the land may be fixed by agreement between public authority and private owner. If a friendly agreement cannot be reached, a commission or board of appraisers is appointed, and a value is established. Of course, the land owner has a right to appeal. He seldom appeals, because he is paid a fair price for the land, and he knows this, if he does not admit it. Now, the land thus taken from the farmer may have produced crops which returned an annual net profit of $15 per acre. The farmer and his heirs had the right to enjoy this profit for an unlimited number of years. Lands that return a net, annual profit of $15 per acre are worth $300 per acre, assuming a rate of 5 per cent. The farmer, under the franchise theory, has the right to demand something because of the right he enjoys to do business for himself. As this right is unlimited as to time, the farmer simply capitalizes his annual, net profit at 5 per cent. This enables him to add another charge of $300 per acre, or $600 in all. We know that the farmer is never permitted to impose on the public in this way, because he is dealing with men, usually his neighbors, who fully understand the value of his land and who also understand the necessities of the public.

After the city has been penalized by the Court for having given the franchise, the corporation adds other charges. One of these has the illusive, yet phonetic, title of "going value." It is true that many private water users pay taxes, but they do not pay taxes on the value of water rights, which they frequently claim to have for sale. They do not pay taxes on the value of the unexpired franchises; they do not pay taxes on going value. These are for sale to the public, not for the public to tax.

Little money is available, usually, to protect the interests of the municipality. The able professional man is tempted to go to the aid of the private party, and he generally does. The sympathy of the Court for the same party may easily be aroused, because the judge, in his professional journey from office boy to the bench, may have lived in an atmosphere hostile to public initiative and unfriendly to the protection of public interests.

In the administration of streams, in the supervision of drainage, in the promotion of public health, and in many similar fields of service, the principles which should control can only be discovered and fully protected by specialists in the various branches. The public rules, which we know as laws, and which should incorporate these principles,

can best be framed by these specialists. The specialist does not invade the domain of the attorney in so doing. Laws relating to these special subjects are, under this practice, couched in good, plain English. They are administered by professional men who understand them. Rarely are they misconstrued. Mr. Johnston.

Wise principles, even when they relate to matters of pure law, do not always develop under the exclusive jurisdiction of the Courts. It may be possible to give one reason for this. When some question, which involves principles of importance to many, is brought to the Court, only a few interested in its solution are able to be represented. As time goes on, many of these find themselves unable to defray the expense of extended litigation, and hence withdraw, so that when testimony is finally presented and argument made, only those who are financially strong remain. The principles which might have been supported by the many who were unable to enter the contest in the first place, and those which were upheld by those who were obliged to withdraw, are never presented to the Court. The decision finally reached must be based largely on the principles supported by the few. This decision probably places the few, fortunate litigants in a position where they may profit, possibly, at the expense of those who were unable to protect themselves in the Court proceeding. When later litigation is indulged in, these fortunate persons are able to meet their rivals in the Court, fortified with greater financial resources and reinforced by a valuable precedent. As time runs on, under such a system, the Court continues to get farther away from the principles which should govern.

When we try to follow the development of underlying principles, we find ourselves immediately interested in men who have led in the various fields of human endeavor. The university has been slow to take an active part in the formulation and discussion of principles. In a way, however, it has prepared men for that service. Only a few years ago, a professional man from the university was either a minister of the gospel, a lawyer, or a doctor of medicine.

As the human race developed, other professions were born, and some of these have grown until they have recognition almost as broad and as general as that accorded the three older branches referred to. Engineering is probably the strongest of the younger professions. It is the province of the engineer to accept the gifts of Nature and to transform them in such a way as to afford the greatest happiness and the highest service to the race he represents. This work is distinct from religion, law, and medicine. In the various branches of engineering activity, principles of great value to the public are discovered by the engineer. These must be observed if the best and highest uses of Nature's gifts are to be protected and enjoyed. The engineer has a great field. He is responsible to the Maker of all things on one side and to the millions of his own kind on the other.

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Four boys grow up together. They enjoy equal educational advantages. They come from homes and from environments that are similar. Each selects one of the four great professions, and proceeds to fit himself to be of service to those with whom he must live, and, incidentally, to support himself in a way that will reflect credit on one who spends years of his life in preparation for work. As years pass, these four men assume high places in their various professions. The lawyer is finally elevated to the bench, the minister becomes a high officer of the church, the physician leads his profession in at least one line of work, and the engineer finds new methods of converting the gifts of Nature into blessings to humanity. Up to this time, these men have been practicing their professions along with many associates. Now, they also become authorities as to principles that concern the work of their own professions. Further, each should be in position to define the principles which should establish the relations of individuals and the public with the things that are studied and understood only by men of their own professions.

When some of these authorities assume an interest in public affairs, there are some obstructions in their paths. A concrete example may serve to illustrate. Assume that the time has arrived for a forward step in matters of public health. The physicians, interested in performing a great public service, naturally advise with the leading men of their own profession. They frame a measure, a duty they are eminently qualified to perform. The measure is submitted to the law-making body. Not a single member of that body represents the medical profession. The committees to which it is referred have no special knowledge of the subject to which the bill relates. Physicians who are active, who are willing to give of their own time for the good of the public, and who have no hope of or desire for reward, appear before these committees. These physicians may not be adept in advancing arguments to those whose training does not lead them to appreciate readily the importance of the pending measure. The committees may finally report favorably. When the measure is placed before the law-makers, the arguments advanced in favor of its passage must come largely from the committees. Its prospects for passage are not bright. If passed, the chances for amendments, which practically nullify the purpose of the bill, are numerous. The bill finally emerges from the legislative test of "running the gauntlet" and becomes a law; but probably is crippled from the beginning. The law probably provides a simple administration. The Governor, or some other officer, far removed from frequent contact with problems of public health, has the appointive power. He may select a competent board of health. He may select an attorney from Podunk, a garbage collector from Utopia, and another politician from Anywhere, as the board to carry this new legislation into effect.

In a short time the entire law may be questioned by some one who claims a "vested" right in maintaining a public nuisance, or in doing things which seriously interfere with public health. Now, this legislation relating to public health, so ably supported by unselfish members of the medical profession in the beginning, is to be taken into the Court, which has had no opportunity to study the principles on which the law is based. It is possible that the great physician who understands the value of the law and who appreciates fully the principles on which it rests, may be called as an expert witness in the initial hearing of the case. Those who have occupied such a place realize how little information such a man is permitted to give, particularly in the definition and support of principles. The witness is confined largely to testimony which might be given by any practitioner. He, better than any man living, is in position to say what principles should govern. An appeal is taken to the Court of last resort, and all testimony is transmitted in documentary form. The great physician is shoved out of the field of service, and the public loses his aid. The great lawyer, the judge in the Court of last resort, never comes into contact with the great physician. He has not followed his work, and he knows as much about the problems of public health as do many others who have not enjoyed close association with them. He cannot appreciate the importance of the principles that the great physician supports, yet he has inherited the right to enunciate the principles that the public is presumed to observe. His decision must be made according to his own light; and there is little new light filtering through to the tribunal of last resort, where problems, so foreign to the law, are considered by the Court. Though we have been laboring for many years under the delusion that a judge, whose training and experience have been confined to the realm of law, may be sufficiently educated, instructed, and informed by testimony relating to a subject, probably technical and at least foreign to him, so that he may make a decision which is equitable and just, still we must condemn the practice when we give it serious consideration. An engineer would not assume that it would be possible for lawyers to make plain to him, in a brief and somewhat stormy contest, the value and importance of great, fundamental principles that underlie problems of pure law. In such a case, the attorney would be the witness. He would be confined to the very elementary phases of the principles under consideration, and, following the prevailing practice of our Courts, he would be bulldozed, persecuted, and villified, while he hesitatingly felt his way along, always knowing that he was not presumed to express his views fully and truthfully and in the light of his special knowledge of the subject. The engineer, acting as judge in this imaginary court, would be unappreciative of the important phases of the questions brought to his attention. He would not be prepared to Mr. Johnston.

Mr. Johnston.

recognize underlying principles of pure law. He would generally be somewhat prejudiced, because some trivial and unimportant matter, relating to his own profession, was indirectly concerned with the really great principles before him. He, therefore, would receive so little help from the witness, and from those conducting the case, that he would have more confidence in his own judgment, usually, than in anything that came to him in the way of testimony and argument.

We can probably ascribe the unfortunate condition of our streams, to-day, to our willingness to permit them to be governed under principles that have been inherited from a decayed civilization or evolved by a profession which is educated, prepared by practice, and finally qualified to perform efficient service in a foreign field.

Finally, a tribute to the great jurists of the past, who studied the best and wisest rules of their own times and then bequeathed to us the benefit of their wisdom and judgment. We have no more valuable inheritance, possibly, than that which has been created by students of the science of law in former years. The principles underlying pure law concern men and their relations with each other. If "the proper study of mankind is man," the study of law is involved therein. Strange as it may seem to some, the legal profession is not so much concerned in general legislation. The Court excuses no man because of ignorance of legislation. There are no professional qualifications prescribed for membership in law-making bodies. Legislation, therefore, is a matter of common concern. It belongs to no particular profession. Speaking generally, other professions than law are supported by principles found in Nature. Man may discover them—he cannot originate them or change them. The principles involved in legislation, which concerns the work of professions of this kind, are all foreign to the profession of law. Injustice is done the Court when it is required to construe such legislation, except where some incidental feature of pure law is concerned. Greater injustice is done the Court when it is required to undertake the administration of things that are largely controlled by principles which are unknown in the science of law. Therefore, when legislation—embracing principles which concern the relation of men to things and to policies foreign to the science of law—is to be framed, interpreted, administered, or scrutinized, this duty should be performed by men educated and trained in those particular branches, more than in law.

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TRANSACTIONS

Paper No. 1322

THE CONSTANT-ANGLE ARCH DAM*

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With Discussion by Messrs. D. C. Henny, Edward Wegmann, Orrin L. Brodie, R. G. Clifford, and Lars R. Jorgensen.

Synopsis.

There are two features which distinguish this type of dam from the ordinary arch dam. These are (1) "Economy of Material", and (2) "Ability of the Dam to act as an Arch, even Close to the Foundation, to a Much Greater Extent than the Ordinary Arch Dam". It is understood that the canyon is wider at the top than at the bottom, which is generally the case.

The first claim, "Economy of Material", is proved in the following manner. It is shown that any arch slice closing the gap between two abutments must subtend an angle of (133° theoretically) about 120° for greatest economy of material.

The constant-angle arch is designed to comply with this law as closely as possible at any elevation from crest to foundation. The length of the up-stream radius is decreased toward the foundation in the same proportion as the canyon becomes narrower, thereby keeping the angle subtended by the arch at the most economical value. In the case of an ordinary arch dam built in a V-shaped canyon, the angle subtended by the arch is rapidly getting smaller toward the lower elevations, and therefore the building material is placed in a less

* Presented at the meeting of September 16th, 1914.

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economical position and the arch soon becomes so flat that it cannot act as an arch, but the load is carried by shear and cantilever action almost exclusively.

To prove the second claim, attention is called to the fact that the deflection of an arch when loaded is proportional to the square of the length of the up-stream radius. This radius may be several times shorter at the foundation than at the crest of the dam in the constant-angle arch design. If the radius should happen to be, say, four times shorter at the foundation than at the crest, the deflection required at the foundation would be sixteen times less than at the crest, using the same unit stresses. In such a case, the arch would be able to support sixteen times as much load at the bottom as an ordinary arch having a constant up-stream radius.

In order to obtain equal safety at all points, it is necessary that the thickness of the arch should be made the greatest in the middle, and diminish toward the abutments, especially toward the lower elevations. This is on account of the fact that an arch dam when loaded acts like a long column in compression, and such a column is weakest in the middle; besides, the central portion of the arch moves back and forth according to different load conditions, and for that reason alone the unit stresses used for this portion should be kept lower than for the more stationary portions of the dam.

It is also pointed out in the paper that the initial stresses (Poisson's ratio), due to the water pressure and to the weight of building material, can be utilized to carry a portion of the load, and that these stresses will carry the greatest proportion of the load in arch dams designed according to the constant-angle arch principle.

An approximate short-cut method for finding the proportion of the load taken by the arch and by the cantilever, respectively, is given, also some information as to the maximum foundation pressure at the toe, and the shear existing at the foundation. Finally, a few practical examples are described.

It has been the object of the writer to bring out a dam design especially adapted to high and comparatively narrow canyons. For such a condition, the constant-angle arch dam will generally show a saving of material of 33% or more over an ordinary gravity dam, and at the same time it will possess a factor of safety more than twice as great as that of the gravity dam.

This method of arch dam design shows how it is possible to make the dam take most of the load, acting as an arch, leaving only a small portion for cantilever or gravity action.

There is no single type of dam which can be readily adapted to every site, therefore a number of types have naturally been developed and applied to satisfy the different demands as these have presented themselves. This statement applies to the type of arch dam to be described in this paper, as well as to any other.

All masonry dams must be built on good rock foundation, with the exception perhaps of those of the Ambursen type, the requirements for which are not so exacting. Where the dam site is narrow and more or less V-shaped, a single arch dam of the type to be described can be built to advantage. How narrow the site must be cannot be stated in general terms, for a few calculations will have to be made to show whether there is economy in using the arch dam rather than any other type. Many dams having gravity sections have been built curved in plan, this curved form giving them considerable extra strength at the crest, especially when the arch subtends a large angle. However, the ordinary arch form gives but little extra strength at the bottom of a V-shaped canyon, because, no matter how large an angle is subtended by the arch at the crest, this angle has become small at the bottom. In other words, the arch cannot carry much load on account of its flatness. This non-action of the arch at the bottom the following method of design seeks to eliminate, besides giving a most economical distribution of material. It does not matter whether a gravity section or a much thinner one is used, the arch can be forced by the design to take up the greater portion of the load. That this is the most economical way of loading the structure should be readily admitted, as arch stress is distributed nearly evenly over the whole section, whereas the stress due to gravity action is very poorly distributed, being a maximum along the down-stream face, and diminishing to approximately zero at the up-stream face. These two stresses are in two different planes, 90° apart, thereby tending to give lateral support to each other, as both act at the same time.* This indicates

* This conclusion can be fairly well proved by the fact that when testing a cube or column of any material in compression, it swells laterally in a plane 90° from the plane of the compressive force. With lead, this is quite apparent. If this swelling be prevented, the crushing strength of the material will be materially increased.

that it is not economy to eliminate entirely the cantilever or gravity action in an arch dam, even if this were possible, as it acts like the hooping in a column, giving lateral support to the dam body. The arch action, however, must predominate, on account of the more economical distribution of stress over the section, and also in order to eliminate the large shear action otherwise introduced by the force causing the bending of the cantilever.

When an arch dam is loaded, the length of the arc is bound to become shorter, due to the deformation of the material caused by the axial stress. The only way the structure can compensate for this shortening is by deflecting down stream. If the canyon has a fairly regular shape, this deflection will be greatest at the crown of the arch and will decrease to zero at or near the abutments. This deflection of the crown introduces cantilever stresses which ordinarily reach their maximum values in the middle section, where the dam is highest and the deflection is greatest. How the load divides itself between the arch and the cantilever depends on their relative ability to carry load, and it is easily seen that if we wish to design an arch dam where the arch is to carry the greater portion of the load, we must keep the arch deflection down to a minimum, and especially at and toward the bottom, where the dam is fixed in position and cannot move. Then, too, the arch should be designed so that the amount of this deflection decreases at a uniform rate from a maximum near the crest to a minimum, as close to zero as possible, at the foundation, in which case any imaginary horizontal arch slice deflects as if it were actually independent and free to move relatively to any other slice above or below, thereby eliminating undesirable internal stresses.

In order to obtain a preliminary dam section for any given dam site the simple formula,

$$t = \frac{P\,R_u}{q} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

can be used for finding the thickness of a sufficient number of arch slices at different elevations. In this formula, t equals the thickness of the dam, in feet, at any given elevation; P equals the water pressure, in pounds per square foot; R_u equals the length of the up-stream radius, in feet; and q equals the average stress, in pounds per square foot, of the area of the dam section (Fig. 1).

It will now be shown how to obtain the most economical arch section, that is, a section where the material is placed so as to take up the load with minimum resulting stress. From the formula, $t = \frac{P R_u}{q}$, it is seen that the thickness, and therefore the area, of the dam section, varies in direct proportion with the radius. The volume of concrete in any arch dam is equal to the area of the section multiplied by the length of the mean arc. The length of the mean arc can be expressed as the length of the mean radius, R_m, multiplied by the subtended angle, in terms of π,

$$\text{or, } V = \text{Area} \times R_m \times 2\,\theta \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where $2\,\theta$ is the subtended angle.

The mean radius, R_m, equals half the width, W, of the span divided by the sine of half the subtended angle (Fig. 1). Thus

$$R_m = \frac{\frac{1}{2} W}{\sin.\, \theta} \ldots . (3)$$

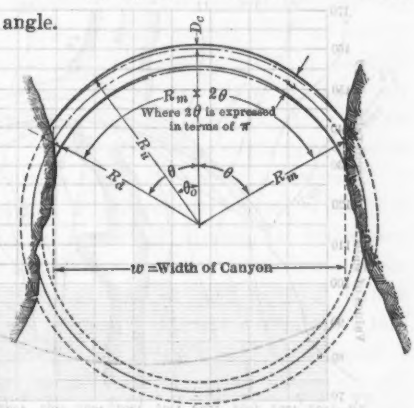

FIG. 1.

As the area of the section is proportional to the length of the radius (both to R_u and R_m), Formula 2, for the volume of masonry can be expressed thus:

$$V = C \times \frac{\left(\frac{1}{2} W\right)^2 \times 2\,\theta}{\sin.^2\,\theta} = \frac{K \times \theta}{\sin.^2\,\theta} \ldots\ldots\ldots\ldots (4)$$

where C and K are constants, the latter depending on the width of the canyon.

According to Formula 4, the volume varies with the term, $\frac{\theta}{\sin.^2\,\theta}$. The differential coefficient of this term, equated to zero, gives the minimum for a central angle of 133° 34′, which means that any horizontal slice of the dam has the least volume when $2\,\theta = 133° 34'$. In other words, the dam contains a minimum quantity of material when the central angle is kept at 133° 34′ at all elevations.

The curve on Fig. 2 shows this graphically; the abscissas represent the central angle, $2\ \theta$, and the ordinates represent the term, $\frac{\theta}{\sin.^2 \theta}$, the latter being proportional to the volume of masonry. In addition to showing the point of maximum economy, this curve also shows that, as long as the subtended angle, $2\ \theta$, is kept between the limits, 150° and 110°, the variation in the volume of masonry is very small, but that, outside these limits, the volume increases rapidly. Fig. 3 illustrates how the dimensions of successive arch-shaped slices of a dam are determined and superposed, one upon the other, to form the structure. Six elevations have been established, forming Contours

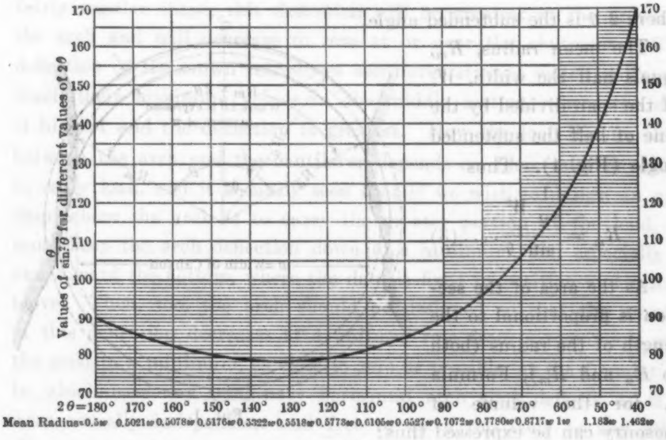

FIG. 2.

I, II, III, IV, V, and VI, Contour VI corresponding to the level of the water retained by the dam, and Contour I corresponding to the lowest water level. The distance across the canyon at the top of the dam, W_6, is measured on a map to scale. Then the mean radius, R_{m6}, which will give the dam the greatest strength with the least volume of material, is found by substituting the most economical central angle, $2\ \theta$, in the formula $R_{m6} = \frac{\frac{1}{2} W_6}{\sin.\ \theta}$. The thickness, t_6, may now be determined algebraically from Equation 1. However, for the first calculation, the length of the up-stream radius will have

to be estimated from the known length of the mean radius, but this is not difficult.

At Contour V the distance, W_5, across the canyon is measured, and the most economical radius and thickness of section at this elevation is found in the same manner as for Contour VI. The center of curvature of the arch slice at Elevation V does not necessarily lie on the same center line as the center of curvature of the arch slice at Elevation VI, as perfectly symmetrical slopes of the canyon sides will rarely be found. It is desirable, however, for practical construction and appearance sake, to try and keep as many centers

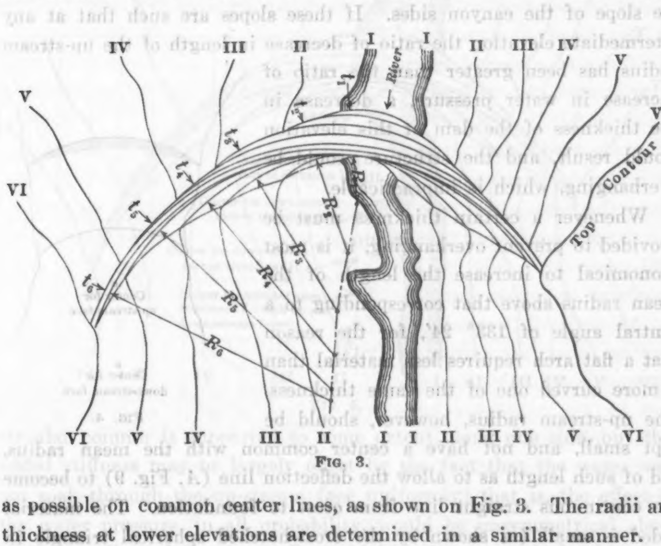

FIG. 3.

as possible on common center lines, as shown on Fig. 3. The radii and thickness at lower elevations are determined in a similar manner. It is thus seen that the main requirement which makes for high economy of material is to keep the central angle of the arch as near to the most economical value at each elevation as the contour of the site will permit. This necessitates the abandonment of the constant up-stream radius generally used in arch dams as at present constructed, and the substitution of radii of varying lengths corresponding to the widths of the canyon at different elevations. In some localities it may be desirable to face the dam with stone, and it may then be of advantage to step the faces. It will then consist of a number of

cylindrical rings superposed one upon the other. The up- and down-stream radii in such cases are ascertained in exactly the same manner as given above.

To prevent upper portions of the dam from overhanging lower portions, it will be necessary to have the thickness of the section increase from the crest toward the foundation. The proportional increase in water pressure, therefore, must be greater than the proportional decrease in length of the up-stream radius toward the foundation. The ratio of increase in water pressure is always fixed, and the ratio of decrease in the length of the up-stream radius depends on the slope of the canyon sides. If these slopes are such that at any intermediate elevation the ratio of decrease in length of the up-stream radius has been greater than the ratio of increase in water pressure, a decrease in the thickness of the dam at this elevation would result, and the structure would be overhanging, which is impracticable.

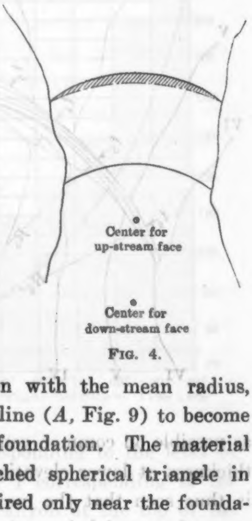

FIG. 4.

Whenever a certain thickness must be provided to prevent overhanging, it is most economical to increase the length of the mean radius above that corresponding to a central angle of 133° 24′, for the reason that a flat arch requires less material than a more curved one of the same thickness. The up-stream radius, however, should be kept small, and not have a center common with the mean radius, and of such length as to allow the deflection line (*A*, Fig. 9) to become one continuous straight line from crest to foundation. The material added is small (as shown by the cross-hatched spherical triangle in Fig. 4), as such additions are generally required only near the foundations, where the arch is short.

In the foregoing, the thickness of the different arch slices has been determined as if all the load were taken by the arch, and the dam section had no gravity action at all. How safe a dam would result depends primarily on the unit compression allowed when using Formula 1 for finding the thickness at different elevations. However, in order to obtain a dam of uniform safety for its whole height, the foregoing design must be somewhat modified to take into considera-

tion the gravity and curved beam action. The foregoing theoretical design would prove to be weakest in the middle in most cases, for the same reason that a long column held at both ends is weakest in the middle, and on account of having the highest cantilever stresses here. Whenever t is small compared with R_u, the arch when loaded is practically a long column in compression, and the length of the arch, therefore, should not be more than twenty-five times its thickness, if the material is to be highly stressed. It is true that this

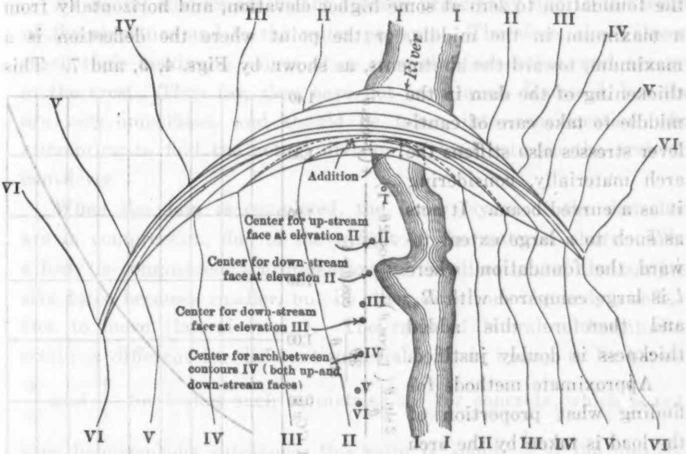

FIG. 5.

circular column is supported to some extent along one side, but this added stiffness may be largely offset by the fact that the water may not soak through the up-stream face uniformly; that is, the effect of the water pressure, in all probability, would be unsymmetrical about the center line of the dam. On a high and comparatively thin arch dam section, the resulting compression due to cantilever action and weight of material above may become excessive near the foundation, requiring some additional material along the down-stream face toward the foundation, as shown by Figs. 5 and 7. On account of the necessity for this additional material, the use of a smaller central angle than the theoretical 133° might actually be more economical, and 120° or even less, in a case like this, might give very satisfactory results (generally greater at the crest and smaller toward the foundation, depending

on local conditions). By decreasing the central angle from 133° to 120°, the volume of material is increased 1% (Fig. 2), but, at the same time, the thickness has been increased 6 per cent. For angles much smaller than 120°, the volume of additional material required increases at a greater rate than the thickness, and therefore a lower limit for the economical value of the central angle is soon reached, and extra material will then have to be added. The thickness of this added material should decrease vertically from a maximum at the foundation to zero at some higher elevation, and horizontally from a maximum in the middle, or the point where the deflection is a maximum, toward the abutments, as shown by Figs. 4, 5, and 7. This thickening of the dam in the middle to take care of cantilever stresses also stiffens the arch materially, considering it as a curved beam. It acts as such to a large extent toward the foundation, where t is large compared with R_u, and therefore this added thickness is doubly justified.

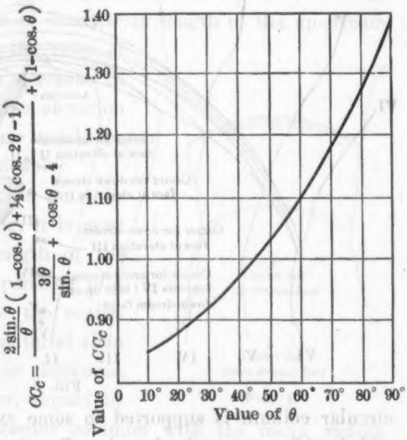

FIG. 6.

Approximate methods for finding what proportion of the load is taken by the arch and by the cantilever have been thoroughly discussed in the paper entitled "Lake Cheesman Dam and Reservoir"* by the late Charles L. Harrison, M. Am. Soc. C. E., and Silas H. Woodard, M. Am. Soc. C. E. The late R. Shirreffs, M. Am. Soc. C. E., in his discussion of that paper develops the following formula for the crown deflection of an arch dam.

$$D_c = \frac{CC_c \times P_1 \text{ (up-stream radius)}^2}{E \times t} \quad \ldots\ldots\ldots\ldots\ldots (5)$$

where $P_1 = P \times \frac{R_u}{R_m}$, and CC_c is a factor which takes the curved beam action into consideration and can be found directly from Fig. 6, which for the sake of ready reference, and on account of a typographical error in the original formula, is reproduced here from Fig. 22 in

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 89.

Mr. Shirreffs' discussion. E is the modulus of elasticity and t is the thickness. This formula and curve, however, do not take the initial stresses in the dam structure into consideration, and, therefore, before applying this formula for finding arch deflections due to the water load, we must determine how much of this load is carried by the arch due to initial stresses, because, only after deducting this part of the load does the remainder divide up between arch and cantilever action in the ordinary sense.

By initial stresses are meant stresses principally due to the weight of the structure and to the water pressure. Therefore, these stresses reach their maximum values at or near the foundation, and are zero at the crest. Thus far, they have not been much discussed, but they are very important, and should be taken into consideration when attempting to find the actual division of load between the arch and cantilever.

When the dam is completed, the lower layers of the dam body are in compression, due to the weight of the material above. When a body is compressed, the dimension in the direction of the compressive force becomes smaller, but in other directions the body swells, if free to move (lateral strain). The ratio of lateral to longitudinal strain is different for different materials. It is known to lie between $\frac{1}{3}$ and $\frac{1}{4}$ for bodies such as metals, but for concrete, which is not a very homogeneous substance, this ratio is about $\frac{1}{5}$ at the unit compression generally used in arch dam design. A. N. Talbot, M. Am. Soc. C. E., gives* values for this ratio, $\frac{1}{m}$, ranging from 0.1 to 0.18 at low loads, up to 0.25 at loads near the ultimate for 1: 2: 4 concrete 60 days old. A French commission on reinforced concrete found Poisson's ratio, $\frac{1}{m}$, for ordinary concrete to be about 0.16 when a force of 200 lb. per sq. in. was applied perpendicularly to the largest dimension of the concrete, and about 0.22 when applied at right angles to the largest dimension. Professor C. Bach, of Stuttgart, Germany, found for Poisson's ratio the value $\frac{1}{5.3}$ for 1 : 2 : 3 concrete 45 days old at a unit compression of about 180 lb. per sq. in.†

* *Bulletin No. 20*, University of Illinois.

† More details may be found in *Engineering News*, Vol. 68, p. 208.

A dam body often contains considerable quantities of large stones, and the presence of these stones has the effect of increasing the value of Poisson's ratio of the total mass. (Poisson's ratio for rock is about $\frac{1}{4}$.) Therefore the use of $\frac{1}{5}$ for this ratio for the total mass seems to be entirely justifiable. Any horizontal layer of material will have to sustain compression corresponding to the height of the masonry above it, and, therefore, will actually become shorter in a vertical direction and have a tendency to expand horizontally. If the abutments are unyielding, the arch may be prevented from actually becoming longer, in which case axial compression is introduced, the same as if water pressure acted on the structure. The value of this initial axial compression per square unit area is equal to the vertical compression multiplied by $\frac{1}{5}$ if the abutments are unyielding, and, as far as the final result is concerned, it does not make any difference whether they are absolutely unyielding or not.

If the specific gravity of the concrete for the dam is taken at 2.3 and the height of the dam at H, then the average vertical pressure can be expressed as $\frac{2.3H}{a}$, where a is the ratio of the total height of the dam to the height of a rectangular wall having the same sectional area and the same base. The ratio, a, is known as soon as the section is known, and in dam design the section must be more or less determined before final calculations can be made.

The dam section shown in Fig. 7 has an area of 9 668 sq. ft., a base width of 70 ft., and a height of 250 ft. The height of the masonry column causing the mean vertical pressure, therefore, is $\frac{9\,668}{70} = 138$ ft., and $a = \frac{250}{138} = 1.81$, making the mean vertical compression on the foundation—in terms of head of water—equal $\frac{2.3\ H}{1.81} = 1.27\ H$, with no water pressure acting on the up-stream side.

The condition of reservoir full introduces an additional force—the radial water pressure—tending to compress the dam body in a direction perpendicular to the direction of the compressive force due to the weight of the body. At the bottom of the dam this force is equal to H, in case the water is standing to the crest of the dam. In this

case the radial water pressure tends to counteract the swelling of the concrete in an up-and-down-stream direction (due to the weight), thereby introducing additional initial axial compression.

The total resulting initial axial compression at the foundation of the section shown in Fig. 7 (in terms of head of water), therefore, is

$$\frac{1}{5}(1.27\ H + H) = 0.454 \times H \ldots\ldots\ldots\ldots\ldots (6)$$

where Poisson's ratio has been taken as $\frac{1}{5}$. Mr. H. Ballet, a French

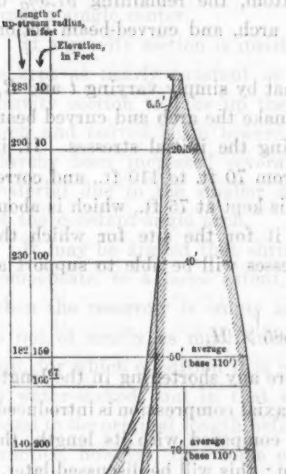

FIG. 7.

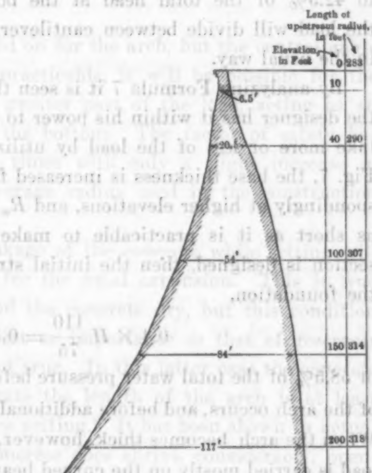

FIG. 8.

engineer, was probably the first to point out the necessity of taking Poisson's ratio into consideration when attempting to find the actual stresses in a dam body.*

The height of water, h, that this initial axial compression of 0.454 $\times H$ will resist without causing any shortening of the length of the arch at the bottom can be found by using Formula 1, thus

$$h = 0.454 \times H \frac{t}{R_u} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII, 1909, p. 51.

For the narrower section shown in Fig. 7, $t = 70$ ft. at the base, $R_u = 75$ ft. Substituting these values, it is seen that this section at the very bottom is able to carry,

$$h = 0.454 \times H \times \frac{70}{75} = 0.425 \times H,$$

or 42.5% of the total head of water, as an arch, before any shortening in the length of the arch occurs.

The initial axial compression holds in equilibrium the stresses due to 42.5% of the total head at the bottom, the remaining 57.5% of the load will divide between cantilever, arch, and curved-beam action, in the usual way.

By analyzing Formula 7 it is seen that by simply varying t and R_u, the designer has it within his power to make the arch and curved beam take more or less of the load by utilizing the initial stresses. If, in Fig. 7, the base thickness is increased from 70 ft. to 110 ft., and correspondingly at higher elevations, and R_u is kept at 75 ft., which is about as short as it is practicable to make it for the site for which the section is designed, then the initial stresses will be able to support at the foundation,

$$0.4 \times H \frac{110}{75} = 0.585 \times H,$$

or 58.5% of the total water pressure before any shortening in the length of the arch occurs, and before additional axial compression is introduced. When the arch becomes thick, however, compared with its length, the load is carried mostly on the curved beam ; this will be discussed later.

The dam shown in Fig. 7 was designed with varying radii to keep the central angle of the arch as nearly constant as possible at all elevations. For comparison, a section is shown in Fig. 8, using the same unit compression (except where it is wider than a gravity section near the foundation) and the same up-stream face batter, but a single common center for both up-stream and down-stream faces. For this section the length of the up-stream radius is also variable, but it increases toward the bottom and reaches here a value of 322 ft.

The average vertical compression on the foundation for the section shown in Fig. 8 is $\frac{2.3}{2.2} H = 1.04 \times H$, and the corresponding total initial axial compression due to the lateral deformation is

$$\frac{1}{5} (1.04 \times H + H) = 0.41 \times H.$$

The height of water which this initial stress can resist, therefore, is equal to (see Formula 7)

$$0.41 \times H \frac{160}{322} = 0.20 \times H,$$

or 20% of the total head. Comparing this with 58.5%, or 42.2% for the constant-angle arch type (Fig. 7), it is easily seen that this latter type is much more effective in utilizing the initial stresses to support the water load than the ordinary arch having its faces struck from a single center.

If a gravity section is insisted on for the arch, but the central angle is kept as nearly constant as practicable, it will be possible for the gravity section to take up the greater part of the load, acting as an arch and curved beam toward the bottom. The factor of safety has thereby been increased several times with only a slight increase in material due to the smaller average radius used in the construction of the constant-angle arch.

It may be argued that shrinkage of the concrete while setting will compensate, to a large extent, for the axial expansion. This is true when the reservoir is empty and the concrete dry, but this condition is not of nearly as much interest or importance as that of reservoir full, for which condition it is not true. In this latter case the concrete is water-soaked, and in that state the length of the arch is at least equal to the original length before setting.* It has been shown in actual practice, however, that some concrete does shrink considerably, opening up contraction joints and cracks. One way of compensating for this, is to force grout into these contraction joints, at a time when the temperature is low and the reservoir empty, after having been filled once. In this way, the permanent set of the structure will also be compensated for, and the arch and cantilever will take their respective shares of the load when the reservoir is filled the next time, and each time thereafter.

Formula 5 has been used for finding the deflection curves, *A* and *B*, of the two sections, Fig. 7 (base 110 ft.) and Fig. 8. Formula 7 has been used for correcting these curves, *A* and *B*, to take the effect of lateral strain into consideration. These curves are shown in Fig. 9, and represent the deflections of the two arches, assuming that they

* See tests reported by Messrs. A. T. Goldbeck and A. H. White at the 1911 Convention of the American Society for Testing Materials.

are free to move at the foundation. They are plotted to show how evenly the deflection curve, *A,* slants from a maximum near the top (above Elevation 40 the two arches are identical, the "horns" being two tangents 40 ft. high) to nearly nothing at the bottom in the constant-angle arch type (Fig. 7), and how little the slant curve, *B,* amounts to in the ordinary type of arch dam (Fig. 8). These curves also show very plainly that from the common arch type much arch action toward the bottom cannot be expected; cantilever and beam action must take the load, because no such deflection as 0.2624 in. could be possible at the point where the arch is fastened to the rock foundation. The constant-angle arch type for this particular site, requiring only 0.0083 in. deflection, or 31.5 times less, to support the same load, will take most of the load on itself, acting as an arch.

For dam sites where the abutments are close to one another toward the foundation, and where t is large compared with R_u, Formula 5 gives values for the crown deflection which are too large, even assuming that the dam is entirely free to move at the bottom. Though this formula considers the curved-beam action, it is at the same time understood that arch action is complete. However, where the arch is thick and the distance between the abutments is short, the arch becomes a wedge, and the horizontal curved beam takes the greater proportion of the load, because, acting in this manner, the support of the same load will require a smaller deflection.

The deflection in the middle of a beam 1 ft. wide, held at both ends, and uniformly loaded, is

$$D_b = \frac{P \times l^4}{E\,J \times 384} \quad \text{(5a)}$$

The notations are the same as before, P being the water pressure, in pounds per square foot; l, the length of the span, in feet; E, the modulus of elasticity of concrete per square foot; and J, the moment of inertia.

Whenever Formula 5*a* gives smaller values than Formula 5, it is indicated that arch action is incomplete. The curved-beam action tends to introduce axial tension along the down-stream face in the middle, and along the up-stream face near the abutments, but the axial compression due to the partial arch action and lateral expansion (Poisson's ratio) will, or should much more than, compensate for this tendency. If it does not, the design should be changed.

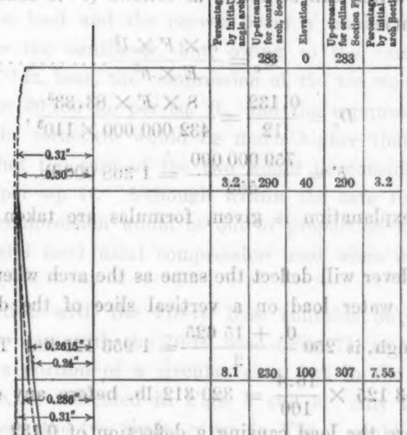
Percentage of load supported by initial stress in constant angle arch section Fig.7
Up-stream radius in ft. for constant angle arch, Section Fig.7
Elevation, Ft.
Up-stream radius in ft. for ordinary arch, Section Fig.8
Percentage of load supported by initial stress in ordinary arch Section Fig.8
283
0
283
3.2
290
40
290
3.2
8.1
230
100
307
7.55
13.2
182
150
314
11.25
16.4
166.67
22.8
140
200
318
15.3
58.5
75
250
322
21
Deflection Curve A
Deflection Curve A corrected
Deflection Curve B
Deflection Curve B corrected
Note: 130' is the mean length of up- and down-stream radius
0.31"
0.30"
0.2615"
0.24"
0.286"
0.31"
0.185"
0.1605"
0.287"
0.324"
0.132"
0.098"
0.0756"
0.279"
0.33"
0.02"
0.0083"
0.2624"
0.328"

FIG. 9.

From Curve *A* the deflection of the arch, Fig. 7, at the one-third point can be ascertained directly, and is found to be 0.132 in. If the cantilever, 250 ft. high and 1 ft. wide, were actually forced to deflect 0.132 in. at this point (Elevation 166.67 ft.) a force, *F*, would be required, which can be found as follows (*F* is concentrated at the one-third point):

$$D_c = \frac{8 \times F \times l^3}{E \times h^3} \qquad \text{(8)}$$

$$D_c = \frac{0.132}{12} = \frac{8 \times F \times 83.33^3}{432\,000\,000 \times 110^3},$$

$$F = \frac{759\,000\,000}{555} = 1\,368\,000 \text{ lb.}$$

(Where no explanation is given, formulas are taken from standard handbooks.)

The cantilever will deflect the same as the arch when thus loaded.

The total water load on a vertical slice of the dam, 1 ft. wide and 250 ft. high, is $250\,\frac{0. + 15\,625}{2} = 1\,953\,125$ lb. The initial stress supports $1\,953\,125 \times \frac{16.4}{100} = 320\,312$ lb. before any deflection takes place, therefore the load causing a deflection of 0.132 in. of the combined arch and cantilever must be equal to $1\,368\,000 + 1\,953\,125 - 320\,312 = 3\,000\,813$ lb. The proportion of this amount taken by the cantilever will be $\frac{1\,368\,000}{3\,000\,813} = 45.5$ per cent.

Now, the actual load to be divided between cantilever and arch is not 3 000 813 lb. per running foot, but only $1\,953\,123 - 320\,312 = 1\,632\,813$ lb. Of this amount the cantilever carries 45.5%, or $1\,632\,813 \times \frac{45.5}{100} = 742\,930$ lb., concentrated at the one-third point, making the actual deflection at this point $0.132 \times \frac{55}{100} = 0.073$ in.

The bending moment due to this force is equal to

$$742\,930 \times 83.33 = 61\,908\,357 \text{ ft-lb.}$$

The section modulus of the base $= \frac{110^2}{6} = 2\,011$, and therefore the compressive stress on the foundation at the toe, due to the bending action of the water load on the cantilever, is equal to

$$\frac{\text{Bending moment}}{\text{Section modulus}} = \frac{61\,908\,357}{2\,011} = 30\,780 \text{ lb. per sq. ft.} \qquad \text{(9)}$$

The total compression on the foundation at the toe will be this compression added to that due to the weight of the structure, which amounts to approximately 16 200 lb. per sq. ft. at the toe, making the total compression approximately 47 000 lb. per sq. ft.

If a base length of 70 ft. is chosen, the arch would take a greater percentage of the load and the curved beam a smaller, leaving the same or less for the cantilever, but, owing to the smaller section modulus of the 70-ft. base, the compression at the toe would be somewhat higher than 30 780 lb. per sq. ft., and the compression due to the weight of the structure would be much higher than 16 200 lb. per sq. ft., so that the sum of the two would be considerably more than 47 000 lb. per sq. ft. Although within the safe limit, the resulting vertical compression would be out of proportion to the 36 000 lb. per sq. ft. (and less) axial compression used when calculating t from Formula 1.

The dam section with the 110-ft. base contains only 4% more material than the dam with the 70-ft. base (Fig. 7), as the addition is not made as a portion of a circular ring, but in the shape of a spherical triangle, as indicated in Figs. 4 and 5. Any intermediate base length between the two limits given in Fig. 7 could be accepted for a dam built on this particular site. (This is shown approximately in Fig. 3, but there the tangents are not indicated.) The two stresses, the 36 000 lb. per sq. ft. average axial compression, and the maximum 47 000 lb. per sq. ft. vertical compression, are acting in planes perpendicular to each other, and therefore tend to support each other. Although they are low, the resulting section (Fig. 7) appears unusually slender on account of the economical distribution of the material.

This method of calculating the vertical stress on the foundation is correct only so long as no tension exists at the heel, or, if tension exists, so long as this tension is properly taken care of. For the constant-angle arch, where the cantilever takes the smaller proportion of the load, there will seldom be occasion for tension along the up-stream face, and there will perhaps never be enough tension to demand consideration. The accuracy of the result obtained from the use of Formula 8 depends to some extent on the face slopes, especially that of the down-stream face. The error, however, is generally such as to compensate for that made in not considering that the width of the

vertical cantilever, which is 1 ft. at the up-stream face, is less at the down-stream face. The short-cut method explained above for finding the division of the water load between cantilever and arch action and that for finding the total maximum foundation pressure, cannot be used for dams having a crown deflection curve similar to Line *B*, as this line does not show a maximum deflection near the crest and a zero deflection at the foundation. The deflection curve, *A*, answers these conditions closely enough for this purpose.

Thus far, only the middle or highest dam section has been considered, as we have been mostly interested in knowing the maximum stresses in the structure, which stresses generally occur (in high dams at least) at the toe, with reservoir full. Some modifications may be necessary near the abutments where the cantilevers, unless very short, do not support their share of the load, most of which is thrown over to the horizontal curved beam and arch, increasing the axial compression on the down-stream face from the abutments to somewhere beyond the points of contrary flexure. These points are located on both sides of the crown, and can be found from,

$$\cos. \theta_0 = \frac{\sin. \theta}{\theta} \dots\dots\dots\dots\dots\dots\dots\dots (10)$$

as shown by Fig. 1. Reference should also be made to the discussion on "Lake Cheesman Dam and Reservoir".* This higher axial compression will not be entirely local, but will be transmitted from abutment to abutment through the middle of the dam, making the axial compression in the middle higher than that due to the proportionate water load carried by arch action; although still lower than the value obtained from Formula 1, assuming the whole load to be carried by the arch. The actual value may be found by drawing a graphical stress diagram, such as used for finding stresses in arch ribs for concrete bridges, etc., keeping in mind that, in the case of a dam, the directions of the forces are radial. From information as to the distribution of the axial stresses thus obtained, a new value for the deflection at the one-third point can be ascertained, and some readjustment between arch and cantilever action can be made. As long as the unit compression used is not much greater than 50 000 lb. per sq. ft., this refinement seems to be unnecessary, and hardly worth the additional labor.

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* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 167.

In Formula 1 only average stresses have been considered in determining the thickness of each individual arch slice. The maximum axial stresses should also be investigated. These exist along the down-stream face, and are found from the formula,

$$Q\text{ (max.)} = q \times \frac{2\,R_u}{R_u + R_d} \quad\text{.......... (11)}$$

Formula 11, however, does not give correct results toward the foundation, where the arch is thick relative to the length of the up-stream radius, and where the span is short. The proportion of the load carried by the arch in such a case is supported more by the curved-beam than by ordinary arch action. This will cause some difference in the value of Q (max.) and q (min.) (as found from Formula 11), adding to Q (max.) at and toward the abutments, and subtracting from it in the middle portion between the points of contra-flexure on the curved beam. In high dams, Q (max.) will ordinarily be lower than the vertical compression at the toe, therefore this vertical pressure is still the most important to investigate.

The influence of Poisson's ratio tends to equalize Q (max.) and q (min.) in dam sections having up-stream faces of steeper slope than their down-stream faces. In such sections the vertical pressure due to the weight of material above is greatest along the up-stream face, and therefore the initial axial compression is also greatest. It is fair to assume that this condition of relieving Q (max.) and adding to q (min.) also tends to improve the water-tightness of the dam.

In all straight gravity dams built across narrow canyons, horizontal tension exists along the down-stream face in the middle, and along the up-stream face near the abutments, at least toward the foundation. This should be very plain when it is considered that any beam fixed at both ends and uniformly loaded will support four times as much load as a cantilever of the same length sustaining the same water load (nothing at the top and a maximum at the bottom). In other words, whenever the beam is four times longer than the cantilever, it will support half of the total load, and whenever this ratio is less than four, the horizontal beam will support most of the load. The ordinary gravity design does not consider this beam action, although, when the dam is built in a fairly narrow canyon, the greatest portion of the load toward the foundation is actually carried on the

horizontal beam, and not on the cantilever. Though adding materially to the stability of the dam (as long as the horizontal tension introduced by this beam action is not greater than the breaking point, and as long as the expansion joints, if any, are placed at or near the points of contra-flexure), the foundation pressure at the toe is at the same time much relieved, a very welcome feature, especially in connection with high dams, and surely this feature should not be left out of consideration when calculating the factor of safety.

Now, if the horizontal beam is curved, horizontal axial compression takes place over the entire section, and the greater the curvature (that is, the smaller the up-stream radius) the more load will be taken by the arch, and the less remains to cause horizontal axial tension at any point of the dam faces, due to beam action. The resultant axial compression from arch action and lateral expansion will in general more than compensate for this tension. Lateral expansion due to the weight of the structure exists, of course, whether the dam is straight in plan or curved, but this alone will seldom be sufficient to compensate for the horizontal tension due to beam action in a straight gravity dam across a narrow canyon. The curvature must be introduced in order to be sure that no tension exists in this horizontal beam. For a dam 250 ft. high, the bottom width of the canyon would have to be well toward ½ mile before a gravity dam would act simply as a gravity section toward the bottom, and before the influence of the horizontal beam action would be negligible, unless it should have failed in tension first. (Near the top, the horizontal beam would have no practical influence.) It would seem logical, therefore, to provide even quite long dams with a curvature sufficient to take care of the horizontal tension. Such dams would not be true arch dams, but the slight curvature would increase the factor of safety by eliminating the tension in the horizontal beam; and combined beam, cantilever, and some arch action would support the load.

Whenever a load is supported on a beam, or on a cantilever, shearing stresses are introduced. These shearing stresses reach their maximum values at the foundation and at the abutments, and should be investigated in order to be sure that they are within safe limits. In the case of the dam section shown in Fig. 7, base 70 ft., it can easily be shown that, even should the shear on the lower 50 ft. of the dam

correspond to the full water pressure, this stress would be entirely within the safe limit, amounting to approximately 4 000 lb. per sq. ft. where the dam joins the hillside and foundation. Along this joint the maximum unit compression generally exists. This compression is so much larger than the shear that actual shear cannot take place, as friction alone will prevent any tendency to sliding at the abutments. Crushing would have to take place before the actual sliding of any element could occur. A fair approximation of the magnitude of the actual shear forces along the joint between the dam foundation and abutments can be obtained in the following manner: Formulas 5 and 5*a* are both applied in order to find in what proportion the load is divided between arch action and curved-beam action toward the bottom. Suppose Formula 5*a* gives, say, half as much deflection as Formula 5 for the same load; then it is indicated that the curved beam carries twice as much load as the arch. In reference to this the factor, CC_c, in Formula 5 should be left out, as we are working close to the foundation, where the arch is short and thick, and where it is therefore necessary to use both Formulas 5 and 5*a*.

From Formula 7 the percentage of the total load carried by the initial axial stresses can be found, and from Formula 8, etc., the percentage of the remaining load (due to head, $H - h$) taken by the cantilever, can be determined. In the case of the dam shown in Fig. 7, this percentage was found to be 45.5, and there is, therefore, 54.5% (of $H - h$) left for the arch and curved beam. Out of this the arch takes $\frac{1}{3} \times 54.5\%$, and the curved beam $\frac{2}{3} \times 54.5\%$ if Formula 5*a* gives half as much deflection as Formula 5. The shear is caused by a force equal to $45.5 + \frac{2}{3} \times 54.5\%$ of the load that remains after that carried by the initial axial stresses (Poisson's ratio) has been subtracted from the total load. Another place where shear action exists is in a vertical plane at the foundation along the toe when the reservoir is full of water. The tipping in a down-stream direction of the loaded cantilever deforms the toe and forces it to make a depression in the foundation, and, as the rock foundation extends outside the concrete toe, shear will take place where the end of the toe tries to deform the foundation. This circumstance introduces additional foundation

pressure at the toe, but this increase is already included in the value as found from Formula 9, at least approximately.*

Before concluding the general discussion, the action of changes in temperature should be considered. As the ends of the arch are fixed to the abutments, the shortening or lengthening of the arch due to temperature changes either causes cracks to develop or forces the crown back and forth. In the constant-angle arch type the average crown deflection for the same amount of decrease or increase in length of arch is only about half of that required by the common arch type (see Curves *A* and *B*, Fig. 9), and the tension necessary to cause this deflection or pulling back of the cantilever, therefore, may not exceed the ultimate strength of the concrete, in which case no cracks would develop. In any event, cracks are not as likely to occur as in the common arch type of dam.

Practical Examples of Arch Dams.

Plates XIII and XIV show the plan and section of the Salmon Creek Dam, 168 ft. high above the river surface, containing 52 000 cu. yd. of concrete, having 1.25 bbl. of cement and a small percentage of lime per cubic yard. The contours represent the actual condition of the site after excavation, and it is seen that the sides of the canyon form an unusually regular V. The crest width of the dam is approximately 550 ft., measured in a straight line, and the arch at this elevation subtends an angle of 113 degrees. Plate XIII shows the location of the centers for various arch slices, 12 ft. apart in elevation, and the length of the corresponding up-stream radii and unit axial stresses are given in the table on Plate XIV. To provide better accommodation for the spillway, the curve for the top 12 ft. of the dam was struck from the same center, therefore the warping of the faces commences 12 ft. below the crest, and continues down to the foundation. This warping is so uniform that one who does not know that the centers are moved constantly in an up-stream direction toward lower elevations does not notice it. The carpenter gets his points about every 10 ft. apart, and it makes no difference to him whether he builds up the

* The result from Formula 9 depends on Formula 8, and in Formula 8 the deflection due to the bending only is considered, not the additional deflection due to the horizontal shear forces accompanying the bending action, which amount to 10% (average), therefore the proportion of load taken by the cantilever, as given on page 702, is really 10% too high, but this makes the resulting foundation pressure at the toe, found from Formula 9, more nearly correct on account of the action of the vertical shear at the toe, and this is the information that concerns the designer most.

PLATE XIII.
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JORGENSEN ON
THE CONSTANT-ANGLE ARCH DAM.

HEIGHT OF DAM ABOVE RIVER BED	CONTOUR	UP-STREAM RADIUS	ARCH STRESS, IN POUNDS PER SQUARE INCH
168'	1175	331'	0
156'	1163	333'	217
144'	1151	325'	283
132'	1139	316'	309
120'	1127	305'	318
108'	1115	294'	320
96'	1103	282'	326
84'	1091	270'	328
72'	1079	257.5'	330
60'	1067	243.5'	330
48'	1055	226.5'	322
36'	1043	207'	309
24'	1031	187.5'	285
12'	1019	168'	256
0'	1007	147.5'	242

face of a cylinder or an inverted cone (approximately). The surveyor, however, has to be more careful than with the layout of an ordinary arch, as, in the present case, there are more calculations to be made and to be followed.

The table on Plate XIV shows that the length of the longest up-stream radius is 333 ft., and the length of the shortest 147.5 ft., the ratio between the two being $\frac{333}{147.5} = 2.26$.

Had the length of the up-stream radius been kept constant, the thickness of the dam at the bottom would have had to be increased 2.26 times for the same axial stresses. Relative to this it should be noted that the arch stresses in the table assume the arch to take the total load, but in reality the stresses are somewhat smaller, as the cantilever takes part of the load. The triangular piece, 10 ft. wide at the bottom (which is added to the lower part of the dam on the down-stream side for the purpose of stiffening the cantilever where it is highest), is not considered in the table giving the arch stresses.

To have kept the enclosed angle (113°) constant at all elevations would have necessitated a greater ratio than 2.26 between the length of the two up-stream radii already referred to. Had this ratio been increased, the structure would have been overhanging in places, and therefore this increase could not be made. This simply shows that it is not always possible to make theory and practice coincide exactly.

To have kept the subtended angle constant in this case would have necessitated a greater bottom width of the site, other conditions remaining the same, but, of course, the dam has to be fitted to the site, and not *vice versa*. This dam creates a reservoir having a capacity of 826 000 000 cu. ft. The drainage area is only 7.5 sq. miles, but the precipitation makes up for it, being more than 100 in. per year.

Plate XIV shows a plan and section of the construction plant, and is self-explanatory. This construction plant, which proved to be the best kind that could be used for this particular place, was laid out under the supervision of Mr. H. L. Wollenberg, who was Chief Engineer of the Alaska Gastineau Mining Company, Juneau, Alaska, the owners, and had charge of this and other work. F. G. Baum, Assoc. M. Am. Soc. C. E., was Consulting Engineer for the Mining Company. The original design of the dam was made by the writer. Plates XIII and XIV, however, were made by the Gastineau Mining Company.

F. C. Herrmann, M. Am. Soc. C. E., Chief Engineer of the Spring Valley Water Company, San Francisco, A. P. Davis, M. Am. Soc. C. E., Chief Engineer of United States Reclamation Service, and the writer visited and reported on the project at various times, and Mr. Herrmann wrote an extensive report thereon.

Figs. 10 to 13, inclusive, show different construction features, thus: The double tower with the two hoppers from which the two concrete chutes distribute the concrete over the dam. These chutes are supported on triangular wooden towers, easily removable. Two ¾-cu. yd. Smith mixers were used, and the progress was at the rate of about 400 cu. yd. daily. Work was shut down for the winter about November 1st, 1913, at which time the dam was half completed. It is expected to have it finished by August 1st, 1914. Indications are that the unit cost will be $7.50, or slightly less, including everything. The electric power plant, depending on the reservoir for its steady supply, is now in operation. Mr. D. C. Jackling is Vice-President, in charge of the operations of the Gastineau Mining Company, and Mr. B. L. Thane is Manager, with offices in Juneau, Alaska.

Plate XV shows the constant-angle arch principle applied on the Lake Spaulding Dam. This dam is part of the South Yuba development, undertaken and owned by the Pacific Gas and Electric Company, San Francisco, Cal.* The lower portion of this dam (60 ft.) is provided with a gravity section for a 260-ft. head, and is arched in plan, the length of the up-stream radius being 600 ft. up to Elevation 4 628. When this elevation was reached (less at the down-stream face, as shown on Plate XV) work was shut down for the winter, 1912-13. During the winter the original plans were changed, and in the following summer the dam was continued to Elevation 4 825, according to the plans on Plate XV. At Elevation 4 628 the length of the up-stream radius was changed from 600 ft. to 250 ft. and kept at this length up to Elevation 4 675. From this level up to the crest, the length of the up-stream radius increases so as to keep the subtended central angle as constant as possible, as shown by the table on Plate XV. This subtended angle is not as large as could be desired, but is as great as the

* This development has been described in detail in articles in *Engineering News*, *Engineering Record*, and others during 1913. Probably the most complete and accurate article on the subject is that written by R. W. Van Norden, M. Am. Soc. C. E., in the *Journal of Electricity, Power and Gas*, San Francisco, December 13th, 1913.

PLATE XIV.
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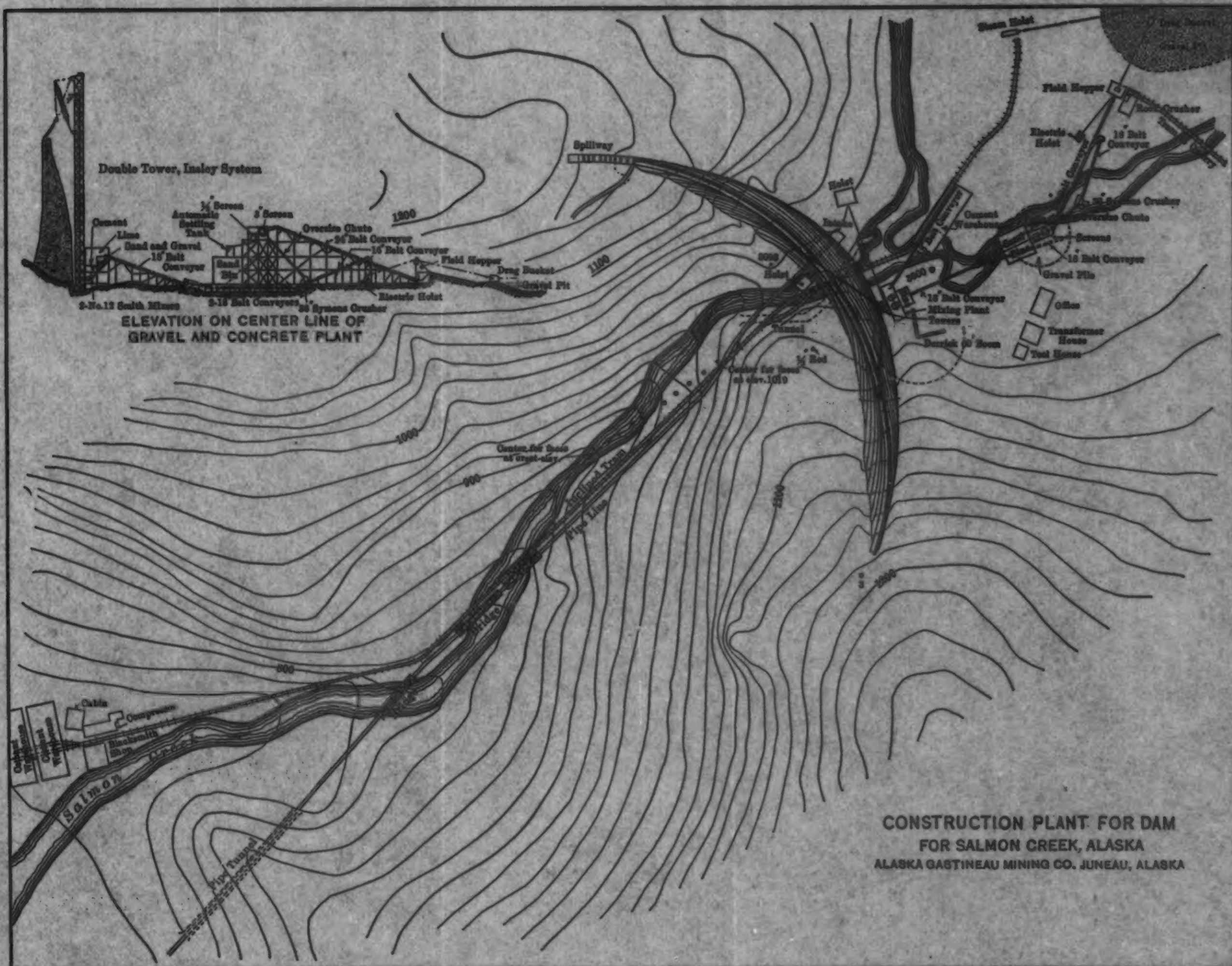
THE CONSTANT-ANGLE ARCH DAM.



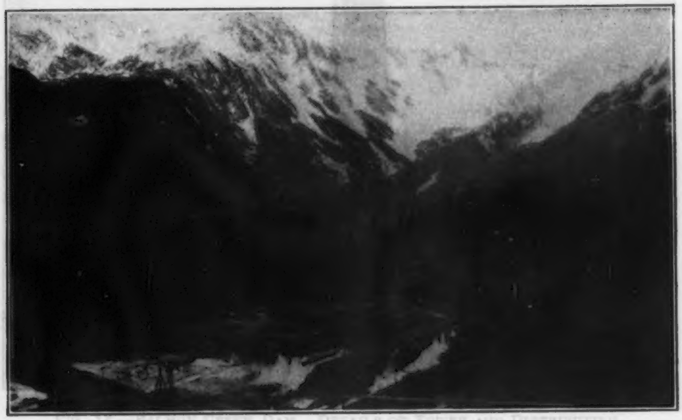


Fig. 10.—Reservoir Site on Salmon Creek, Alaska. Looking Up Stream from the Dam. Gravel Pits in the Foreground.

Fig. 11.—Salmon Creek Dam, Showing Construction Tower and Form Work.

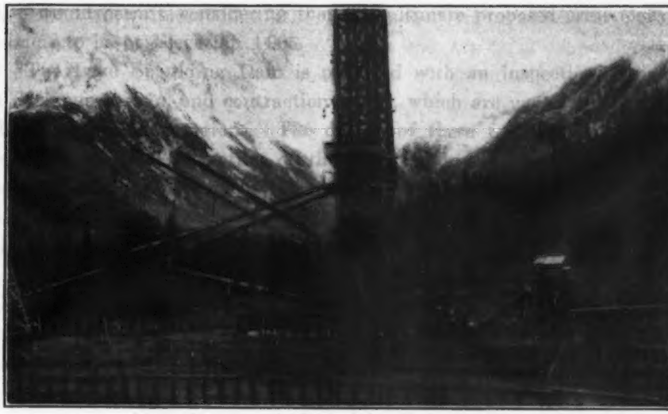


Fig. 12.—Salmon Creek Dam. Details of Tower and Distributing System for Concrete.

Fig. 13.—Salmon Creek Dam. Construction Plant.

site would permit, considering that the ultimate proposed crest elevation is to be at Elevation 4 905.

The Lake Spaulding Dam is provided with an inspection tunnel, a drainage system, and contraction joints, which are usual features in dams of large proportions. The details of these features are shown on Plate XV. The section of the arch above Elevation 4 660 is of such dimensions that it will stand an extension of 35 ft. in height above the present crest elevation (4 825) without any addition to its thickness. The maximum arch stress (q in Formula 1) will exist at Elevation 4 775 with the water level at Elevation 4 860, or 260 ft. above the river bed, and will amount to 23.8 tons. It is fairly constant over the greatest portion of the structure, as can be seen from the table on Plate XV.

When the time comes to extend the crest of the dam to Elevation 4 905, 305 ft. above the river bed, a slab of concrete must be added to the down-stream face, and, in order to effect a good bond between the present dam and the new slab, the down-stream face of the present dam has been stepped off and a sufficient number of iron rods have been left protruding several feet to grip the new slab and hold it in place.

The Lake Spaulding Dam was constructed of a wet mixture of gravel concrete. Toward the top this mixture was richer than at the bottom, and contained from 1.17 to 1.25 bbl. of cement per cubic yard. Test specimens were broken every day, and the results of these tests were used as a guide in fixing the exact proportion of the mix. It was required that the breaking strength of the test specimens (8 in. in diameter) should be at the rate of 400 lb. per sq. in. when 7 days old; and 900 lb. per sq. in. when 28 days old. Toward the lower elevations, where the arch was thick, the mixture was somewhat leaner. The concrete was mixed in four 1-cu. yd. Smith mixers on the hillside above the dam, and, after being turned in the mixers for 1½ min., it was run by gravity in flumes to different portions of the dam, as shown on Figs. 14 and 15. After the dam had risen to a certain height the concrete could flow no more by gravity to portions farthest away, and a 30-in. belt-conveyor system was installed. With this system, all portions of the dam could be reached. There were also two cableways, to transport lumber and other materials to the dam. During the latter part of 1913 there was

very little water in the reservoir, the outside temperature was low, and the chemical heat was practically out of the dam body. This had the effect of opening up the contraction joints (which are 80 ft. apart) about $\frac{1}{8}$ in. In addition to this, cracks about $\frac{1}{16}$ in. wide, appeared midway between each contraction joint. These were all along radial lines, the same as the contraction joints. When the water in the reservoir rose to the crest, early in February, 1914, these cracks closed.

The outlet works from the reservoir consist of two intake tunnels (shown on Plate XV), concrete lined, and of a finished diameter of 8 ft. 8 in. One intake is at Elevation 4 670; the other is 100 ft. above. Each intake is provided with a 72-in. butterfly valve. The upper intake slopes downward about 48° until it meets the lower tunnel, this slope starting a few feet back of the upper butterfly valve. About 1 000 ft. down stream the single pressure tunnel ends in an adit, and is there provided with a second butterfly valve and also with two pressure reducers. Later, it is intended to install a 5 000-kw. turbine and let this act as a pressure reducer by utilizing whatever head there may be in the reservoir. From this point the 350 sec-ft. of water will flow by gravity toward the power-houses below, of which one is built, and four more are projected. After the water has left the lowest power-house, it will be used for irrigation.

The actual construction work was started under the direction of the late James H. Wise, Assoc. M. Am. Soc. C. E. It was continued and completed to date with F. G. Baum, Assoc. M. Am. Soc. C. E., as Chief Engineer. John R. Freeman, A. P. Davis, and H. F. A. Schussler, Members, Am. Soc. C. E., and others acted as Consulting Engineers at different times. Mr. Freeman, working in conjunction with Mr. Davis, made an extensive report, and many of the details suggested by these gentlemen are incorporated in the construction of the dam. Mr. Davis visited the site several times during the construction period. In the actual design as used, the writer's arch theory, and the shape of the up-stream face suggested in designs submitted by him, have been followed, except near the foundation, where an ordinary conical face was substituted below Elevation 4 675, instead of continuing the inverted cone (approximately) above.

Mr. John A. Britton was General Manager and Vice-President of the Company, H. C. Vensano, Assoc. M. Am. Soc. C. E., Civil Engineer, R. G. Clifford, Assoc. M. Am. Soc. C. E., Designing Engineer

PLATE XV.
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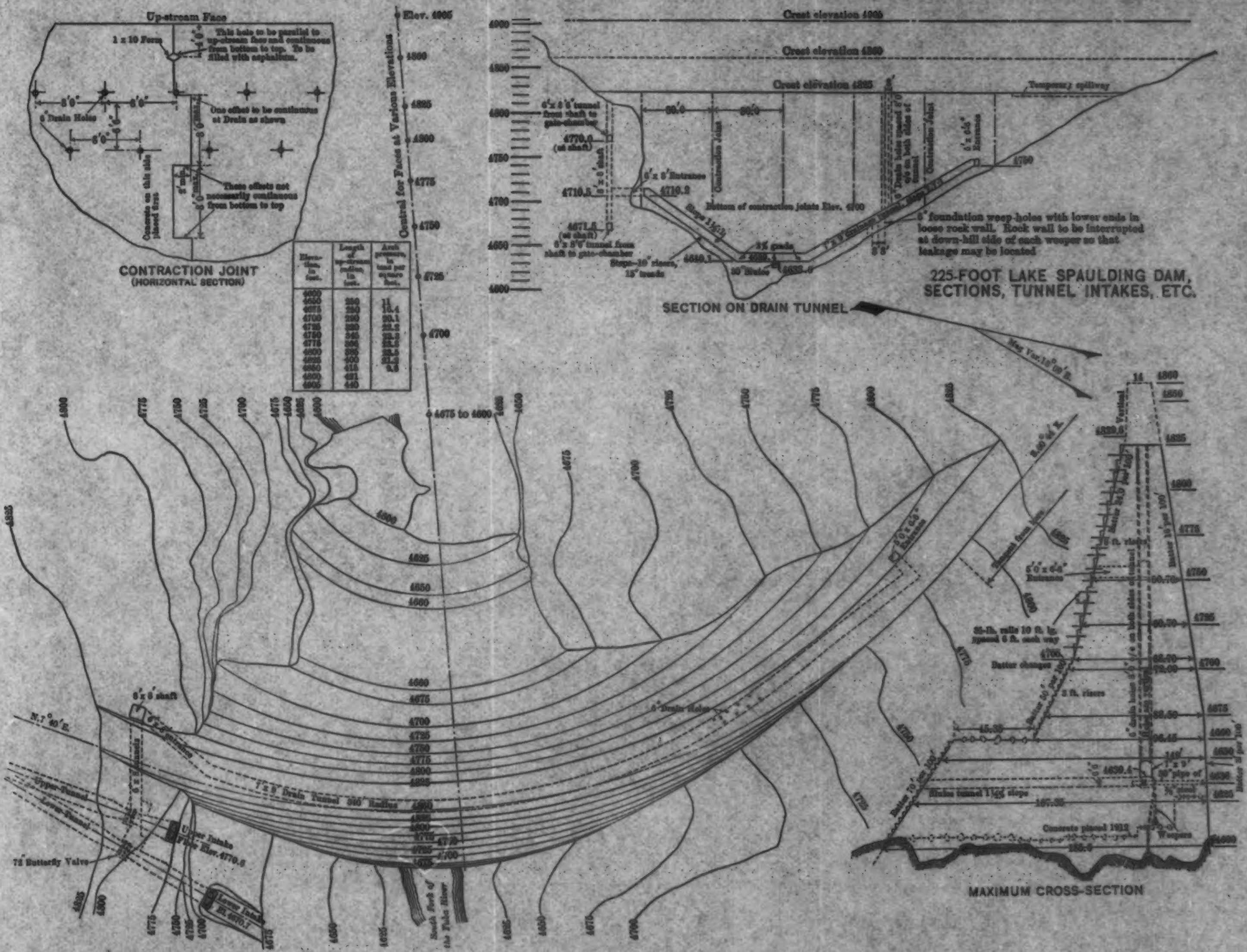

225-FOOT LAKE SPAULDING DAM, SECTIONS, TUNNEL INTAKES, ETC.

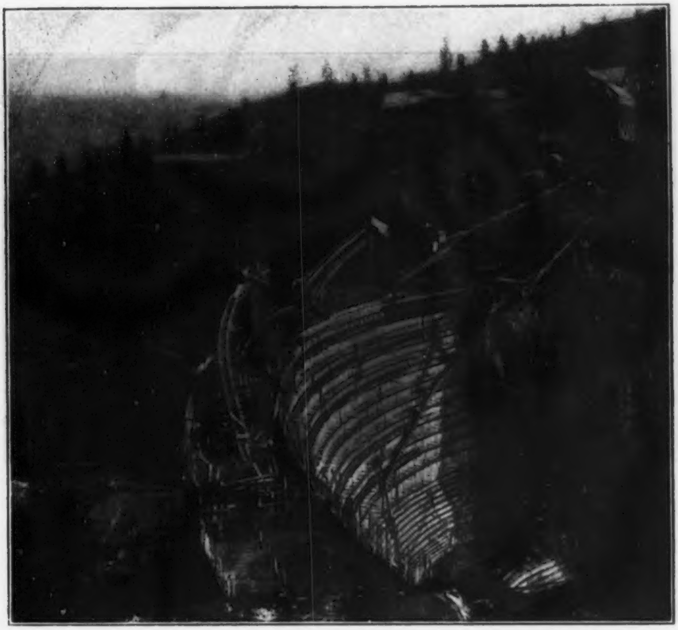

FIG. 14.—LAKE SPAULDING DAM. MIXING HOUSE, DISTRIBUTING FLUMES, AND BELT CONVEYORS. THE METHOD OF CONSTRUCTING THE EXPANSION JOINTS CAN BE SEEN.

FIG. 15.—DOWN-STREAM VIEW OF LAKE SPAULDING DAM, SHOWING THE STEPPING-OFF AND IRON RODS FOR BOND.

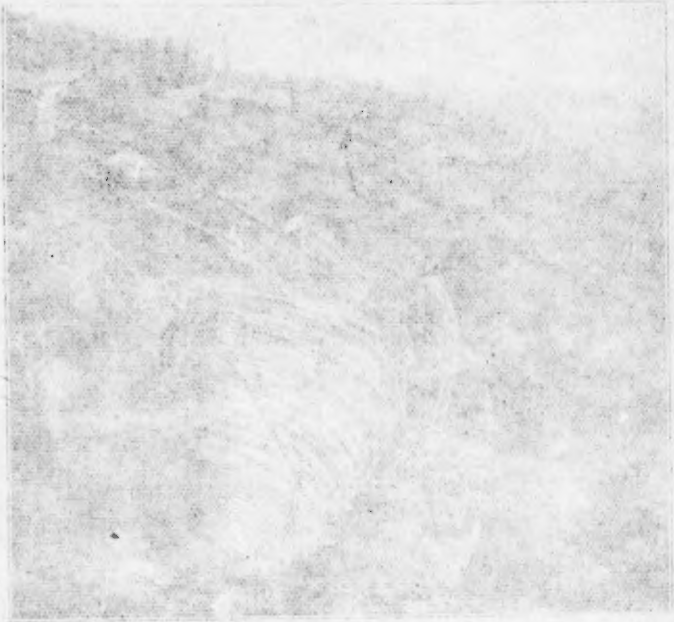

Fig. 16.—Down-stream Face of Lake Spaulding Dam, 225 Feet High Above River Bed.

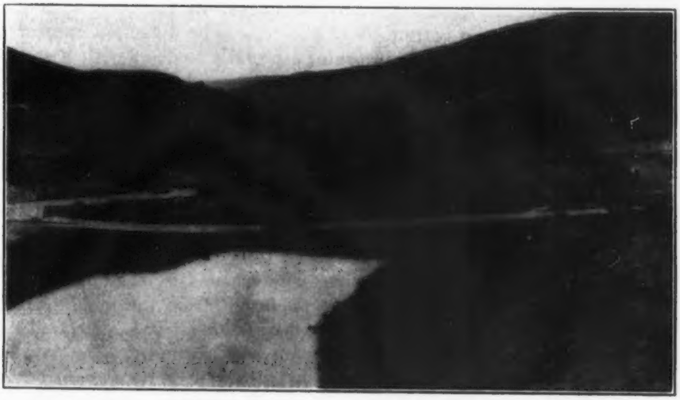

Fig. 17.—A Small Overflow Dam. Showing Obstruction in the Middle to Facilitate the Admission of Air Behind the Sheet of Falling Water.

FIG. 16.—[illegible] OVERFLOW DAM, [illegible]

FIG. 17.—A SMALL OVERFLOW DAM, SHOWING CONSTRICTION IN THE MIDDLE TO FACILITATE THE ADMISSION OF AIR UNDER THE SHEET OF FALLING WATER.

and Resident Engineer when the dam was started. Mr. P. Magerstadt was Resident Engineer during 1913, and Messrs. Duncanson and Harrelson superintended the construction.

An arch dam 30 m. (98.4 ft.) high, with a base width of 4 m. (13.12 ft.) and a crest width of 1 m. (3.28 ft.), having an up-stream radius of 20 m. (65.6 ft.) at the bottom and 35 m. (114.8 ft.) at the crest, has been designed and built by H. F. Cameron, M. Am. Soc. C. E., Division Engineer of the Bureau of Public Works, Manila, P. I., to store water for domestic use.*

Fig. 17 shows a small diverting overflow dam designed on the constant-angle arch principle. It is built at an elevation of 13 000 ft. above sea level, in the Peruvian Andes, and is part of a hydraulic development by the Cerro de Pasco Mining Company. Messrs. F. G. Baum and Company, of San Francisco, were the engineers, and Mr. H. L. Wilcox was the Superintendent. This dam is designed to take care of a maximum overflow of 1 500 sec-ft., and is 62 ft. high, from bed-rock to crest, but only 24 ft. high above the river bed. The length of the up-stream radius at the crest is 36 ft., and at the river bed, 18 ft.

* *Engineering Record*, August 23d, 1913, p. 203.

DISCUSSION

Mr. Henny.

D. C. HENNY,* M. AM. SOC. C. E. (by letter).—The economical design of arch dams in canyons of considerable width at the bottom and with practically vertical sides, calls for a constant angle and constant radius. The author deserves full credit for pointing out that in canyons which are more distinctly V-shaped the design requires a variable radius and as far as possible a constant angle in order to obtain maximum economy.

It does not need much mathematical analysis to be convinced of the general correctness of his reasoning, and the writer is in part responsible for the adoption of a similar design for a dam of this type, now under construction.†

With the more detailed reasoning in the author's analysis of interior stresses throughout the body of masonry, however, the writer cannot fully agree, largely because he does not believe that the facts correspond entirely with the assumption on which the analysis is based.

It has been assumed by the author, as is customary in studies of this kind, that the weight of the dam is transferred vertically downward to the foundation, and that the water pressures against the structure are transferred horizontally to the abutments. A division of the loading is thus arrived at between the vertical cantilever and the horizontal arch, and Poisson's ratio is further applied in an attempt to approximate more closely actual stresses and deformations.

This method of reasoning may be logical in the case of a dam almost as long at the bottom as at the top. The principal subject of the paper, however, is believed to refer especially to dams with V-shaped rather than U-shaped longitudinal sections, and to such dams this reasoning is not believed to be logically applicable.

In the matter of the distribution of foundation pressures due to gravitation alone, a difficult problem is presented in which the shape of the canyon and the method of construction become important factors.

If a dam is built as a monolith and brought up horizontally, with the layers well connected or interlocked by large rocks, and if the sides of the canyon are steep, the dam becomes to a large extent a wedge, and the load will be partly transferred from the middle to the sides of the canyon through combined arch and beam effect. Thus, the loading at the bottom of the canyon will become greatly reduced.

If, on the other hand, the dam is built with vertical contraction joints, the horizontal load transference will become reduced, although friction along such joints, if under heavy pressure through lateral

* Portland, Ore.

† Clear Creek Dam, Washington.

expansion due to vertical loading and through horizontal water loading, will still leave considerable beam and arch effect. Mr. Henny.

If the dam is built up in vertical slices with large vertical joints between, and these joints are finally filled, under the most favorable conditions of temperature, not only will lateral pressures from vertical loading be almost completely nullified, but entirely different conditions as to initial stresses will exist.

The effect of the shape of the canyon and the difference in methods of construction will be equally important as to transference of horizontal water loading.

If Fig. 18 represents the developed longitudinal section of an arch dam, it is evident that in a dam of that type, the shortest distance of a point *a*, in the mid-vertical cross-section, to the abutment does not lie in a horizontal, but in an inclined direction, approximately normal to the slope of the canyon. A down-stream movement of the point *a*, such as may result from water pressure, therefore, will produce relatively more deformation and compression in the direction, *ab*, than in the direction, *ac*.

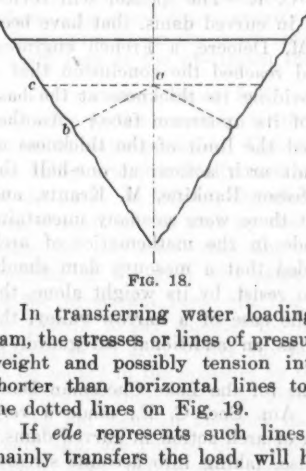

FIG. 18.

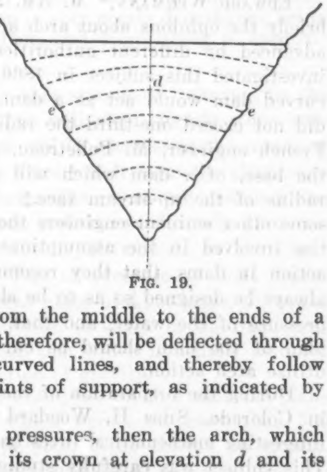

FIG. 19.

In transferring water loading from the middle to the ends of a dam, the stresses or lines of pressure, therefore, will be deflected through weight and possibly tension into curved lines, and thereby follow shorter than horizontal lines to points of support, as indicated by the dotted lines on Fig. 19.

If *ede* represents such lines of pressures, then the arch, which mainly transfers the load, will have its crown at elevation *d* and its springing lines at the lower elevation, *ee*, and if developed will approximate an elliptical shape, with largest radius of curvature and with least thickness at the crown.

If these pressure lines are representative of stress distribution, it is evident that there will be an increase in intensity of pressure near the top of the middle section and a corresponding decrease at the bottom.

Mr. Henny.

If the dam is not built as a monolith but with contraction joints, there will be a lesser tendency to deflection from the horizontal; but, through friction in connection with initial and loading stresses, much of this tendency will remain, and it is quite probable that even then the pressures in the lower part of the dam, both up stream and down stream, may be profoundly affected by load transference.

In any study of division of load between arch and cantilever, moreover, it should be remembered that any inclined plane from the canyon walls to the top of the dam may develop a section which, considered as a cantilever, may, because it is shorter, be stronger and, therefore, have greater effect in opposing true arch action than the vertical cantilever on which the author's reasoning is based.

To summarize, the writer holds that if an analysis of interior stresses in a dam placed in a V-shaped canyon is to yield satisfactory results, it is essential that the section of the canyon along the center line of the dam should be made a factor in the problem.

This discussion is submitted with a full appreciation of the principal point brought out in the author's valuable paper.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—The speaker will review briefly the opinions about arch action in curved dams, that have been advanced by different authorities. M. Delocre, a French engineer, investigated this subject in 1866, and reached the conclusion that a curved dam would act as a dam, providing its thickness at the base did not exceed one-third the radius of its up-stream face.† Another French engineer, M. Pelletreau, placed the limit of the thickness of the base, of a dam which will permit arch action, at one-half the radius of the up-stream face.‡ Professor Rankine, M. Krantz, and some other eminent engineers thought there were so many uncertainties involved in the assumptions made in the mathematics of arch action in dams, that they recommended that a masonry dam should always be designed so as to be able to resist, by its weight alone, the pressure of the water, and that, in the case of a narrow valley, the plan of the dam should be curved, as an additional safeguard, to utilize arch action.

During the preparation of the plans for the Lake Cheesman Dam, in Colorado, Silas H. Woodard, M. Am. Soc. C. E., made a very interesting mathematical investigation of arch action in curved dams.§ This subject was carefully studied, also, taking into account stresses caused by changes of temperature, by George M. Wisner and Edgar

* New York City.

† "Mémoire sur la forme du profil à adopter pour les grands barrages en maçonnerie des reservoires," par M. Delocre. *Annales des Ponts et Chaussées*, 1866, 2e Semester, p. 212.

‡ "Mémoire sur les murs qui supportent une poussee d'eau"; par M. Pelletreau. *Annales des Ponts et Chaussées*, 1876, Octobre; 1877, Août et Novembre.

§ "Analysis of Stresses in Lake Cheesman Dam"; by Silas H. Woodard, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIII, p. 108.

Mr. Wegmann.

T. Wheeler, Members, Am. Soc. C. E., in connection with the plans for the Pathfinder Dam.*

Mr. Jorgensen has taken a step in advance in the study of curved dams, by showing that a vertical arch, built between the sides of a narrow valley, will require the minimum volume of masonry when it has a central angle of 133° 34′. In applying this principle to the design of the profile of a curved dam, he divides the dam into a number of assumed courses, and tries to make the central angle of each course about 133° 34′. If this principle were carried out rigorously, some of the higher courses might overhang the lower ones. The author proves, however, that the volume of masonry required in any course will not vary much if the central angle lies between 110 and 150°, and, by allowing this latitude in the central angles of the different courses, he obtains a rational, economic design, which differs from the usual plan for curved dams by having different radii for the up-stream face at different levels. The method is novel, and is theoretically correct.

The author states, on page 687, that:

"Arch stress is distributed nearly evenly over the whole section, whereas the stress due to gravity action is very poorly distributed, being a maximum along the down-stream face, and diminishing to approximately zero at the up-stream face."

The speaker cannot fully accept this statement. Owing to the elasticity of the masonry, a curved dam will yield slightly when subjected to water pressure. This motion will be greatest at the center of the valley and practically zero at the sides which form the abutments of the arch. It follows, therefore, that although we can calculate the compression in the arch by the simple, well-known formula given on page 688, this stress may not be uniformly distributed, as the slight yielding of the arch will cause the line of pressure to shift nearer to the up-stream face. In the present state of knowledge, it would be difficult to say whether the distribution of stress in the dam, considered as an arch, varies any less than that which is usually assumed to be caused in the masonry by the action of gravity.

Turning from theory to practical examples, we find that the plans of the old Spanish masonry dams, which were built more than 300 years ago, are curved or polygonal in plan, being convex up stream. About 1843, before any correct theory had been advanced for the design of masonry dams, M. Zola built a curved masonry dam to form a reservoir for the water supply of the City of Aix, in the southern part of France. This dam, known as the Zola Dam, is so thin that it cannot resist the water pressure by its weight. Its stability is solely due to arch action. For a height of 120 ft. above the founda-

* *Engineering News*, August 10th, 1905, p. 141.

Mr. Wegmann. tion, it has a width of only 42 ft. at the base. The dam is 205 ft. long on the crest, and is curved to a radius of 158 ft. at the top.

In 1884, Mr. F. E. Brown built a curved masonry dam in the Bernardino Mountains in California, which equals the Zola Dam in boldness. It was 64 ft. high and only 20 ft. wide at the base. At a depth of 48 ft., the line of pressure for reservoir full was about 13 ft. outside of the dam. This structure stood successfully until 1910, when it was replaced by a safer dam, about 200 ft. farther down stream.

From 1896 to 1906, Mr. Leslie Augustus Burton Wade built thirteen curved, concrete dams in New South Wales, Australia.* They vary in height from 25 to 87 ft., and their masonry is subjected by arch action to maximum pressures of from 20 to 25 tons per sq. ft. The highest of these dams has a width of base of 24 ft. for a height of 87 ft.

At the end of his paper the author mentions three dams which have been built successfully on the constant-angle arch principle. The profiles of these structures are rather peculiar, but this is due to the author's attempt to get a better distribution of stresses and a smaller volume of masonry than are to be found in the ordinary curved dam.

Mr. Brodie. ORRIN L. BRODIE,† M. AM. SOC. C. E. (by letter).—The author is to be congratulated on the generally clear and concise treatment of the subject matter of his valuable paper and on his demonstration of the constant-angle arch principle. The paper has proved of particular interest to the writer as he has had occasion several times in the last few years to analyze the stresses in arched dams of different types. He gladly greeted the author's statement, early in his paper, of his presentation of a short-cut method for finding the distribution of the total load between arch and cantilever elements of the structure. The writer had developed a set of general formulas (including the moment of inertia of the horizontal section of the cantilever portion considered between vertical radial planes), so that a direct solution was possible for a dam of any vertical cross-section; but the actual computations prove long and tedious. The formulas referred to were developed along the lines laid down by the late R. Shirreffs, M. Am. Soc. C. E. In connection with this approximate short-cut method, the writer, not having opportunity to try it for himself, would like to know whether it is applicable to a dam of any cross-section, say a spillway dam (at least 120 ft. high), arched, and which requires many changes, particularly at the top and bottom, in slopes of its down-stream face. He would also like to know what expenditure of time would be required in determining completely, by the ingenious short-cut method of the author, this distribution for a dam of the indicated type.

* *Minutes of Proceedings,* Inst. C. E., Vol. 178, p. 1.

† New York City.

The author mentions a typographical error in the original of the arch deflection formula of the late Mr. Shirreffs, relating to a trigonometric function of θ. The derivation of this formula, it is true, will result in the expression as written by the author in connection with Fig. 6, but it was originally written, evidently, to retain the angle, θ, throughout, instead of involving 2 θ, as the author has written in the numerator of the first term of the expression for CC_c. That is, the last term of this numerator, $\frac{1}{2}$ (cos. 2 θ — 1) is equal to (cos. $^2\theta$ — 1), as originally expressed. Mr. Brodie.

The author's Formulas (6) and (7) are open to some question, in the writer's mind.

The intensity of stress due to the weight of a large dam, reservoir empty, is not generally uniformly distributed in an up-and-down-stream direction, over the foundation, though for a thinner arch ring this uniformity of distribution may be approached. Furthermore, the initial axial stress (in terms of head of water) due to the weight of the dam, assuming for the time that its weight is uniformly distributed over the base or foundation, will, with Poisson's ratio taken at $\frac{1}{5}$, be as the author states, $\frac{1}{5} \times 1.27\ H$, where H is the height of the dam above the horizontal layer at the foundation level. Unyielding abutments are supposed to resist this axial strain; but the resulting strain in the horizontal direction at right angles to the axial strain, up and down stream, is opposed by the water pressure only on the up-stream side of the dam at the given level. Yet, with the water level at the top of the dam, a head of $\frac{1}{5}\ H$ is assumed to oppose this up-and-down-stream strain. The unrestrained down-stream side has been neglected in considering $\frac{1}{5}\ H$ as in Formula (6), by the author.

In other words, unless there be a force on the down-stream side, opposing the water pressure on the up-stream side, it is not clear to the writer how the initial axial stresses can be considered to resist the intensity of pressure due to a head of water, h, nor is it at all clear how this "resistance" can be so definitely determined, as stated in Formulas (6) and (7), with the above mentioned facts in mind.

In closing, the writer wishes to emphasize the thought expressed by the author that there is no single type of dam that may readily be applied to every site. Each site provides its own problem to be thoroughly investigated.

Mr. Clifford.

R. G. CLIFFORD,* ASSOC. M. AM. SOC. C. E. (by letter).—Having developed this type of dam, the author has also demonstrated its theoretical features so thoroughly and forestalled possible practical objections so ably, by introducing and answering them himself, that little room is left for discussion until such time as actual measurements of deflection or of stresses in dams of this type may bear out or refute his theoretical deductions. There is one gratifying feature, namely, that the safety factor used in computing the arch stresses is sufficient to make the design safe, even if one omits the proportionate parts of such stresses which presumably are relieved of their work both by the cantilever action and the resistance of the so-called "initial axial stresses."

When it is attempted to combine properly the varying elements of stress discussed as working together for the respective support or overthrow of dams of this type, no one would be so hardy as to base the safety of a community living below a reservoir created by such a dam on a close mathematical solution without large factors of safety, until practice has demonstrated their soundness. In one plane is the curved beam action, amounting, in some cases, to nearly pure arch stress, and again introducing considerable tension and shear. Also, in this plane, are the initial axial stresses depending on the characteristics of the particular concrete used, and varying with the unit weight of concrete and water at the level considered. There must also exist in this horizontal plane internal stresses caused by the swelling of the concrete as the material is subjected to varying degrees of moisture and heat. In the vertical plane are the counter forces of cantilever action and possible hydrostatic uplift against the weight of concrete and water.

Just how all these forces divide up the loads imposed, the writer claims can seldom be approximated, for here may enter the unknown variables, such as contraction and expansion due to setting or wetting, which can upset the other calculations utterly. A contraction due to cooling and drying has been measured under similar conditions, and amounted to ½ in. per 100 ft., and this contraction, in an arch 600 ft. long, with a radius of 400 ft., is equivalent to a deflection of about 2.7 in., which, in the dams referred to by the author, would exceed considerably the total cantilever deflection up to the point where tension is introduced in the heel of the dam. This means that the arch has little or no chance to act, as it was calculated it should, in order to help out the vertical forces, for the dam has virtually been divided into short curved blocks which cannot act together until the water pressure has tipped them together radially, or until the concrete has absorbed sufficient moisture to swell and close the cracks.

* Sacramento, Cal.

The practical solution of this problem would seem to be a thorough grouting of contraction joints after the concrete has finally set and thoroughly cooled, and while the reservoir is empty. This would insure that the dam would act as an arch as soon as any water pressure was exerted against it. Mr. Clifford.

The writer desires to add a few facts concerning the Spaulding Dam, which is one of the first examples of this particular type and is described briefly in the paper. This dam is in a narrow canyon of bare, seamless granite, which converges down stream at the site, making some form of arch logical for any type chosen. The dam was started as a gravity structure, with a 400-ft. radius and practically a vertical up-stream face.

In the spring of 1913, the more economical type under discussion was decided on, but the necessity for building on the foundations already in place made it necessary to introduce some modifications of the true constant-angle arch dam. The up-stream batter required in the middle of the dam to prevent overhang at the horns, forced the top of the dam down stream to such an extent as to fit the ground poorly, and, in order to overcome this, the curves on the up-stream side were made to end in tangents, and the down-stream radii were extended to their respective contours, thus thickening the dam at the abutments which are full gravity sections at the points of tangency. It was also deemed unnecessary to reduce the radii of the up-stream face below 250 ft., because at this point (60 ft. above the base) the width and thickness were about the same. This condition precluded the possibility of pure arch action, the resulting wedge acting entirely as a curved beam.

The present dam, 225 ft. above bed-rock, has a maximum possible arch stress of only 17 tons per sq. ft., and the top can be raised 35 ft. more, adding 50% to the present storage without increasing these pure arch stresses to more than 24 tons, which is considered a conservative figure for the material used. The bottom 60 ft. was built as part of a 260-ft. gravity structure, and, to show the economy of applying these new principles, this same base is ample to provide for carrying the dam up to a height of 305 ft. above the base, at which time this lower portion will have no possible arch stresses greater than 14 tons per sq. ft., although the thickened base in a high dam is advisable to distribute the high crushing values due to cantilever action, as expounded by the author.

The determination of the ultimate economical height of the dam was simplified by the fact that, just beyond the point where the reservoir would no longer be filled during the winter of a low year, the cost of construction became prohibitive, owing to other extensive damming along a series of subsidiary outlets. The unit cost of additional storage to the company's old system became a minimum of $22

Mr. Clifford.

per acre-ft. with the main dam crest 285 ft. above bed-rock, but to increase this height to 305 ft. the cost only increased to $24 per acre-ft., still leaving a reasonable cost of development with a reservoir that could be counted on filling every year. The value of this stored water is realized when it is noted that the series of power developments below this reservoir will utilize a total vertical drop of 4 000 ft., the water then being in great demand for irrigation and domestic use in a well-populated, fertile community.

The original estimate of 315 000 cu. yd. for the main dam is cut down now to 270 000 cu. yd., a reduction of 15%, and it is only fair to Mr. Jorgensen to state, that if his design could have been used in its entirety, a still further saving of 10% could have been effected, with only a slight increase in stresses near the base, which are now very low, as already noted.

It is earnestly to be hoped that the engineers of the companies which are the pioneers in building arch dams of this modified and economical type will take measurements of deflection under varying heads and at regular intervals around the dams, in order that actual results may bear out, or throw new light on, the present theoretical treatment. The Spaulding Reservoir filled rapidly a month after the dam's completion, when 80% of the concrete was not yet 6 months old. At this time of filling, the deflections shown in Table 1 were observed at the center of the dam's crest, which is 600 ft. long and 20 ft. thick, with a radius of 400 ft., tapering to a tangent at each end.

TABLE 1.—DEFLECTIONS IN SPAULDING DAM.

	Elevation of water below crest, in feet.	Deflection in crown of arch, in inches.
January 4th, 1914............	40	2.62
January 7th, 1914............	30	2.62
January 28th, 1914..........	7	3.33
February 10th, 1914.........	5	3.44

The zero of the deflections was set on November 25th, with water 175 ft. below the crest. It is quite conceivable that, as the setting was completed and drying out proceeded, the internal molecular action attendant on contraction, had actually drawn the crown of the arch down stream before the water rose, and a series of readings taken later, as the water dropped, would be enlightening.

The author gives the approximate extent of contraction cracks on radial lines 40 ft. apart (joints were left 80 ft. apart radially) as alternately $\frac{1}{8}$ and $\frac{1}{16}$ in. on January 4th, which is about the same as the writer's independent estimate, no accurate measurements being taken except those of an ordinary rule. Later, these cracks partly closed as the water pressure increased and the concrete swelled, but

it would seem advisable to grout them during the late fall, while the concrete is still dry and cold, so as to work the arch action more and the cantilever less, otherwise heavy compressive strains will develop along the down-stream face, upsetting what would be a well-distributed series of stresses. Mr. Clifford.

LARS R. JORGENSEN,* M. AM. SOC. C. E. (by letter).—It is possible in most cases, to dimension the length of the up-stream radii, on a constant-angle arch in a V-shaped canyon, in such a manner that the crown will deflect in a straight line, this deflection being a maximum toward the crest and nearly zero at the bottom. Mr. Jorgensen.

If this cannot be done, the crown will deflect nevertheless practically in a straight line by transferring stresses from one elevation to another through shear and bending. It is also very likely that some transference is done, as suggested by Mr. Henny, in Figs. 18 and 19, in all cases. The customary allowable arch pressure, varying between, say, 15 and 25 tons per sq. ft., is sufficiently low to take care of some local concentration of stress.

Although French engineers have been responsible for much of the development work in dam design, statements made by some of them, as to the relation which ought to exist between thickness of dam and length of up-stream radius at the bottom, can only be taken as expressions of their personal opinions.

As to the question whether or not a gravity section ought to be used for the arch, no general answer can be given. Where the project will stand the expense of large safety factors, the dam, of course, should be designed with at least the same factor of safety as that used on the other structures of the same project. This, perhaps, necessitates the use of low arch stresses, and, consequently, a heavy section is the result, but the material should be placed to the advantage of the arch, as this form of structure is the most economical for closing a reasonably short and deep gap.

As to the sentence with which Mr. Wegmann takes issue, the writer begs to state that in certain cases it might be possible that the actual difference in unit compression between the up- and down-stream faces will be the same for an arch and a gravity dam, but, if calculated on a percentage basis, the distribution of stresses in the arch will be the more even of the two. For instance, an arch calculated for an average stress of 25 tons per sq. ft. may have this stress distributed in the ratio of 30 tons at the down-stream face and 20 tons along the up-stream face near the abutments, due to the curved beam action and the great thickness of the arch. If this distribution is compared with that in the gravity section having 10 tons unit compression along the down-stream face and zero tons unit compression along the up-stream face, the actual difference would be the same in each case,

* San Francisco, Cal.

Mr. Jorgensen.

but, calculated on a percentage basis, the distribution of the arch stresses is seen to be much the better.

In the middle of an arch dam the curved beam action tends to neutralize whatever difference there would be otherwise between the unit compression along the two faces, if the proportions of structure and site are such that this action takes place to a considerable extent. The difference between the maximum and the minimum arch compression in the middle of the dam (center of canyon), therefore, is small, although toward the abutments it may be quite large. Toward the abutments, however, there is little or no movement of the dam, and, therefore, higher local compressive stresses can be tolerated than toward the middle portion of the dam, which moves back and forth with the rise and fall of water level and temperature.

To Mr. Brodie's question as to whether the method given for finding the proportion of the load taken by the arch and by the cantilever, respectively, is applicable to his spillway arch dam, the writer can only state that it is applicable for arch dams having deflection curves similar to Curve *A* of Fig. 9, and, in order to have an arch deflect thus, it is necessary that its up-stream radii decrease in length from the crest toward the foundation; this is what makes a better arch.

The formula in Mr. Shirreff's discussion, referred to in the paper, is correct, but the writer preferred to use Mr. Shirreff's expression (9*b*) as the type in the final formula did not seem plain. Mr. Brodie's point, as to whether the full water pressure, H, in Formulas (6) and (7) should be taken, or only a percentage, when considering the influence of Poisson's ratio, is well taken.

It is true that the water pressure on the up-stream face is not opposed directly by any opposite force, but it is opposed, however, by two reactions directed through the two abutments; therefore the full head, H, of water has been substituted in Formula (6), as it has also been used by modern French engineers. When working with concrete, Poisson's ratio cannot be regarded as an exact fixed quantity, $\frac{1}{5}$; therefore, a smaller error in the determination of H in this matter cannot be regarded as very serious.

It has not been claimed that the formulas given were sufficient for furnishing complete information as to all possible internal stresses, bending, shrinkage, etc. Even in the simplest steel roof truss, such stresses are not possible of determination, but are left to the factor of safety to take care of.

As concrete is a more uncertain material than steel, a concrete dam will require the selection of a higher factor of safety than would be necessary for a steel roof truss. With an arch stress of 24 tons per sq. ft., there is a large margin of safety, even with the existence of the unknown and variable internal stresses. Some concrete shrinks

considerably while setting and aging, especially when deposited at a high rate of speed; this probably accounts for the opening of the contraction joints and intermediate cracks on the Lake Spaulding Dam when in an unloaded condition. The arch cannot pull the crown back when the structure is being loaded, but must act together with the cantilever, the resulting deflection being a function of the modulus of elasticity of the body. Mr. Jorgensen.

The arch consists practically of separate voussoirs, each approximately 40 ft. long, and, therefore, it can only resist compressive forces. It will be interesting to know how much the crown will deflect the second time the reservoir is filled, as by that time the structure will have taken its permanent set and will be completely aged.

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TRANSACTIONS

Paper No. 1323

STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS

Discussion.*

BY MESSRS. A. C. JANNI AND WILLIAM CAIN.

Mr. Janni. A. C. JANNI,† M. AM. SOC. C. E. (by letter).—In a recent paper by William Cain, M. Am. Soc. C. E., entitled "Stresses in Wedge-Shaped Reinforced Concrete Beams", there were enunciated formulas for computing the maximum compressive stresses in concrete, as well as the maximum tension in steel. These formulas, together with the assumptions underlying them, are of such a nature as to lend themselves to a critical study which, the writer thinks, may not be entirely devoid of interest.

Leaving out of consideration the points of minor importance in this paper, it is thought expedient to confine this critical analysis to the principal ones. In the first place, in dealing with the problem referred to, the author, in simplifying a certain expression, uses a method somewhat objectionable.

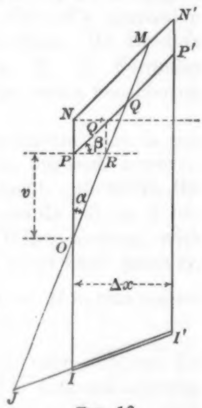

FIG. 16.

Fig. 16 is similar to the author's Fig. 2 (*b*), and shows two parallel sections, $N I$ and $N' I'$, of a wedge-shaped reinforced concrete beam. Let O be the position of the neutral axis of the section, $N I$, and $M J$ the position of $N I$ after stress; the angle of rotation being α.

A foot-note on page 748 of the paper is as follows: "In the triangle, $P O Q$, by the law of sines, $P Q = \dfrac{v \sin. \alpha}{\cos. (\alpha + \beta)}$. As α tends toward zero, this approaches $\dfrac{v \alpha}{\cos. \beta} = v \alpha \cos. \beta$."

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* This is a continuation of the discussion on the paper by William Cain, M. Am. Soc. C. E., published in *Transactions*, Vol. LXXVII, p. 745.

† New York City.

This simplification is not correct. A mathematical simplification cannot be performed on a mathematical abbreviation. Now, cos. $(\alpha + \beta)$ is purely a symbolical expression, and one cannot operate on it without developing first the expression itself. Mr. Janni.

Cos. $(\alpha + \beta)$ is not an actual value, and can be an actual value only when α and β have assigned numerical values, which is not so in this case.

The correct simplification of this expression is represented by:

$$P\,Q = \frac{v\,\alpha}{\cos.\,\beta - \alpha\,\sin.\,\beta}$$

which cannot be simplified further, when one considers that $v\,\alpha$ and α sin. β are infinitesimals of the same order, and that the previous expression is not final but must pass through an integration, as is shown in the paper.

It may be readily seen that the author, with that simplification, did not find the value of *PQ*, as he thinks, but of a part of it only.

In fact, in the triangle, *POR*, Fig. 16, α being very small, $PR = \alpha\,v$; and RQ' being parallel to *OP*, or perpendicular to *PR*, $PQ' = \frac{\alpha\,v}{\cos.\,\beta} = \alpha\,v\,\sec.\,\beta$; therefore, the value, $\alpha\,v$ sec. β, does not represent the length of *PQ*, but that of *PQ'*, which is a part of *PQ*.

It may be demonstrated also that the difference, *QQ'*, is precisely due to the erroneous simplication.

In fact, in the triangle, *PRQ*, with the notations in Fig. 16 and by the law of sines:

$$R\,Q = \frac{v\,\alpha\,\sin.\,\beta}{\cos.\,(\alpha + \beta)}.$$

Similarly, in the triangle, *RQQ'*:

$$Q\,Q' = Q\,R \times \frac{\sin.\,\alpha}{\cos.\,\beta} = \frac{\sin.\,\alpha\;\;v\,\alpha\;\;\sin.\,\beta}{\cos.\,\beta\;\cos.\,(\alpha + \beta)} = \frac{\alpha^2\,v}{\cos.\,(\alpha + \beta)}\tan.\,\beta.$$

Therefore:

$$P\,Q = P\,Q' + Q'\,Q = v\,\alpha\,\sec.\,\beta + \frac{\alpha^2\,v\,\tan.\,\beta}{\cos.\,\beta - \sin.\,\beta\,\alpha}$$

$$= \frac{v\,\alpha}{\cos.\,\beta - \sin.\,\beta\,\alpha}$$

which is the value of *PQ*, as previously stated by the writer.

It seems to the writer that in the paper referred to, as well as in its closure, there is a confusion of conception in regard to the behavior of a toe-beam, a dam, and counterfort.

A dam, when properly built, may be designed,* as far as the maximum compressive stress is concerned, considering a vertical element of it 1 ft. thick and assuming that, for a certain height, it is a vertical

* F. Platzmann, "Ueber den Querschnitt der Staumauern." Leipzig, 1908.

Mr. Janni.

beam, fixed at the base and free at the top, under the actions of its dead load and of the hydraulic pressure against its up-stream face.

The writer does not think it necessary to enter into any analysis concerning the design of a dam; this important kind of construction now has a rather complete theory, which does not need any workable formula for its design.

A toe-beam, supporting a retaining wall, etc., may be designed as a horizontal beam, if its shape is as in Fig. 1,* fixed at one end and free at the other, under the actions of the vertical reaction of the soil and of the friction developed along the base line.

Concerning now the case of a counterfort, as shown in Fig. 17, the writer does not understand on what assumptions it is designed as a beam.

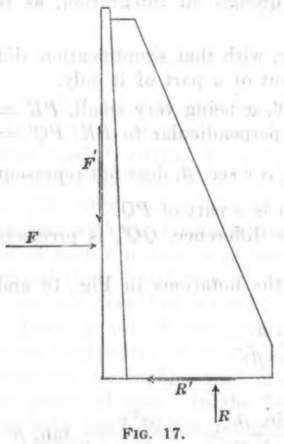

FIG. 17.

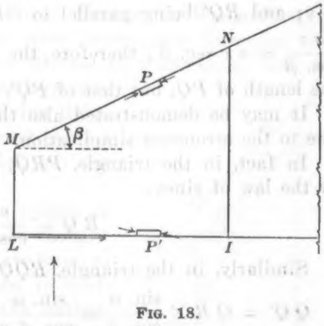

FIG. 18.

If a beam, as commonly understood, means a solid having one dimension a great deal larger than the other two, and supported or fixed at both ends, or fixed at one end, having the other end either free or supported, it is clear that the solid shown in Fig. 17 is not a beam.

That solid is under the actions of the horizontal thrust along its height, as well as of the reaction at the base and of the friction against the soil; there is no moment at the base which would entail the fixed-end hypothesis.

The author is quite right in demonstrating that the maximum compressive stress along *MN*, Fig. 18, is parallel to *MN*, and the writer begs leave to borrow that demonstration, in order to show that in the same beam the maximum tensile stress is not parallel to *LI*, if friction is included.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 745.

For convenience the writer will quote the author's demonstration, and alongside of it will place the demonstration supporting his own statement: Mr. Janni.

Author's demonstration that the direction of the stresses at any point of the upper surface is parallel to *MN*.

"* * *; for, consider a small rectangular parallelopiped of the concrete at *P*, with faces parallel and perpendicular to *MN*. There can be no shearing stress on *MN*, as no part of the beam extends above *MN* to produce shear, and there is no external force acting on *MN* except the atmospheric pressure, which does not exert any friction on the face. It must follow that there can be no shear on the planes at right angles to *MN*, as unit shears on planes perpendicular to each other are equal. Consequently, the compressive stress on a plane at *P*, at right angles to *MN*, is normal to it, or parallel to the face."

Writer's demonstration in support of his statement that the maximum tensile stress at any point of *LI*, is not parallel to *LI*.

In fact, consider a small rectangular parallelopiped of the concrete at *P'*, with faces parallel and perpendicular to *LI*. There is shearing on *LI* developed by the friction due to the soil reaction. It must follow that there is the same shear on the planes at right angles to *LI*, as unit shears on planes perpendicular to each other are equal. Consequently, the maximum tensile stress on a plane at *P'* at right angles to *LI*, is not normal to it, nor is it parallel to the face.

The writer's conclusion might have been reached by other considerations, but it was thought that it would be more interesting to the reader to reach it by using the same truths as those used by the author.

The author, however, does not think so, and, after a rather long discussion which is all but a mathematical demonstration, concludes that "it has been rigorously proved that 'the maximum tensile stress at *I* is parallel to *NI*' whether or not friction is included". It is the writer's opinion that this demonstration does not demonstrate anything as to the direction of the tensile stress along *LI*.

It is difficult to understand the purpose of the demonstration concerning the well-known continuity of stresses in a beam, brought forth so elegantly by Culmann by means of his "circular diagrams".

When it is assumed that all compressive stresses, with respect to the section, *NI*, are parallel to *MN*, it is going too far.

From the description in the paper of the behavior of those stresses along *NI*, the writer has the impression that the author, perhaps, refers to "principal" or "ideal stresses", instead of bending stresses, as usually called.

If the writer's assumption is correct, then it cannot be assumed that they are parallel, neither can it be assumed that, with respect to the

Mr. Janni. section, NI, plane sections before stress remain plane sections after stress.

Theory clearly shows the behavior of those stresses and the shape of NI after stress.

Fig. 9* shows the lines of the ideal or principal stresses in the toe, $MNIL$, under the action of the vertical reaction. This figure may be obtained, for instance, by cutting Fig. 13† at 4-4 or 5-5.

Obviously, those stresses, with respect to NI, are not altogether compressive stresses, but, at each point of NI, the component of it, normal to NI, will be the compressive stress, and the component of it, parallel to NI, will be the shear at that point, this being quite an elementary matter.

Speaking of compressive stresses not being normal to a cross-section is nonsense. Furthermore, the stretching of the fiber, PP', with respect to the section, NI, is a function of the modulus of elasticity, E, as well as of the coefficient of Poisson, and not of the modulus of elasticity, E, only. However, whatever may be the meaning of the paper concerning the nature of those internal forces, the writer thinks that the author entered a rather difficult field when he tried to devise a workable formula.

The mere fact that the shear at N is never zero, as a consequence of the method of design shown in the paper, should have warned the author that something was wrong with his method.

Following his method, the shear at N is a maximum when $\beta = 45°$; that is to say, at N, if $\beta = 45°$, there is maximum compression and maximum shear simultaneously.

The author, however, states: "Above the neutral surface, it decreases, by the usual parabolic law, to zero at N."

Formula 8‡ for the computation of the maximum compressive stress in concrete, may also be written (see Fig. 10§):

$$f_c = \frac{2\,M}{b\,y\left(h - \frac{1}{3}\,y\right)\,\cos.^2\,\beta}$$

where: f_c = maximum compressive stress in concrete,
M = external moment,
β = angle of inclination of the top of the beam to the horizontal,

the other letters having the meaning shown in Fig. 10.

This formula shows that, if N keeps a constant section, and MN rotates about N, the angle, β, increasing gradually, while M keeps constant also, the NI becomes weaker and weaker.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 765.
† *Loc. cit.*, p. 771.
‡ *Loc. cit.*, p. 751.
§ *Loc. cit.*, p. 758.

Mr. Janni.

Neither does the limitation of β to 60° for the permissible use of the author's formula prevent the foregoing absurd conclusion; nor should his remark, that the line, MN, should be extended to the left of the section, NI, be taken into serious consideration.

It is well known that the left part of that solid remains there only and solely in order to carry the action of the external force, which is acting on it, on the section, NI; provided the acting external force has constant intensity, direction, and distance from the section, NI, its moment (external moment) will not change in the least its value, whatever be the geometrical shape of the solid on the left of NI.

If the author, in dealing with wedge-shaped beams, as in Fig. 18, had considered the cross-section at N normal to MN, instead of parallel to the acting forces, he would have avoided absurdities, and would have seen that, after all, theory is not "plainly inadequate".

The author says:

"Suppose a reinforced concrete T-beam, Fig. 15, with an inclined flange; would not a designer naturally take the compressive stresses in the flange as acting parallel to its surface?"

To this the writer would like to reply that a designer would take the compressive stresses as the author says; but he would not consider those compressive stresses as acting on the vertical section, as shown in Fig. 15; if he did, he would be flatly wrong.

The case of a toe-beam, as in Fig. 18, is not one of bending only, but of bending and compression at the same time.

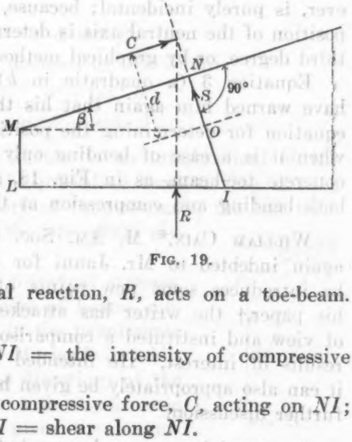

FIG. 19.

Fig. 19 shows how the vertical reaction, R, acts on a toe-beam. In this figure:

$C =$ Component normal to $NI =$ the intensity of compressive force acting on NI;

$C \times d =$ External moment of the compressive force, C, acting on NI;

$S =$ Component parallel to $NI =$ shear along NI.

It would be rather interesting to know what are the "mathematical difficulties in the way where" a section "is not parallel to the line of action of the loads".

The writer has reason to believe that, in the location of the neutral axis in a cross-section of a wedge-shaped beam, the elementary principles, which are the basis of such determination, have been ignored.

In fact, assuming that all stresses have the same direction and

Mr. Janni. that plane sections before stress will remain plane sections after stress, is the same as assuming that the stresses are proportional to the distances from the neutral axis, no matter what the common direction of those stresses may be.

In other words, the center of compression, being fixed in position, as well as the center of tension, the position of the neutral axis is fixed also, no matter what may be the intensity of the stresses (within the elastic limits) and their direction.

The center of compression, after all, is the antipole of the neutral axis with respect to the ellipse of inertia of the compressed part of the section, and the center of tension is the antipole of the neutral axis with respect to the ellipse of inertia of the part of the section under tensile stress.

This relation is purely a graphical one, and as long as the centers of compression and tension do not change their position, that of the neutral axis cannot change.

The statement, therefore, that the plain methods used to locate the neutral axis in a cross-section of a prismatic beam "are entirely inapplicable to a wedge-shaped beam" is quite incorrect. This remark, however, is purely incidental; because, in the case under discussion, the position of the neutral axis is determined either by an equation of the third degree, or by graphical methods.

Equation 3 (a quadratic in k), obtained by the author, should have warned him again that his theory was astray; for, a quadratic equation for determining the position of the neutral axis is obtained when it is a case of bending only; but, in the case of a reinforced concrete toe-beam, as in Fig. 18, as has been said already, there is both bending and compression at the same time.

Mr. Cain. WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The writer is again indebted to Mr. Janni for a stimulating discussion in which he introduces some new points of view. Since the publication of his paper,† the writer has attacked the subject from another point of view and instituted a comparison of methods which leads to some results of interest. He intended to publish this independently, but it can also appropriately be given here in connection with Mr. Janni's further discussion.

Mr. Janni devotes much space to the expression, $P\,Q = \dfrac{v \sin. \alpha}{\cos. (\alpha + \beta)}$. According to the writer, as α tends indefinitely toward zero, this expression tends toward the value, $\dfrac{v\,\alpha}{\cos. \beta} = v\,\alpha \sec. \beta$. The writer is unable to catch Mr. Janni's view-point in objecting to this statement,

* Chapel Hill, N. C.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 745.

Mr. Janni contending that cos. $(\beta + \alpha)$ does not tend to cos. β when α tends to zero. A number of proofs will be given, in the hope that one, at least, may meet his objections. Mr. Cain.

(1) From the unit circle, Fig. 20, in which, $P\ C\ M = \beta$, $M\ C\ N = \alpha$, cos. $(\beta + \alpha) = C\ A$, cos. $\beta = C\ B$, we see that as α tends toward zero, N approaches M, A approaches B, and $C\ A$ approaches $C\ B$ indefinitely, which proves that cos. $(\beta + \alpha)$ approaches cos. β indefinitely as α tends toward zero. Further, cos. $(\beta + \alpha) = C\ A$, is as much a "value" as cos. $\beta = C\ B$.

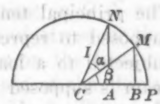

FIG. 20.

(2) Cos. $(\beta + \alpha) =$ cos. β cos. α − sin. β sin. α. As α tends toward zero, the right member tends toward cos. $\beta - 0 =$ cos. β. Otherwise, sin. β sin. α is an infinitesimal and can be neglected in comparison with the finite first term, cos. β cos. α, which by the same theory of infinitesimals can be replaced by cos. β.

(3) From Fig. 21, $R\ Q = O\ R$ tan. α, $P\ Q = R\ Q$ sec. $\beta = O\ R$ tan. α sec. β; therefore, as α approaches zero, Q approaches P, $O\ R$ approaches $O\ P$, and tan. α approaches α; whence $P\ Q$ approaches indefinitely the value,

$$O\ P.\ \alpha \text{ sec. } \beta = v\ \alpha \text{ sec. } \beta.$$

(4) Finally, take the expression deduced by Mr. Janni:

$$P\ Q = v\ \alpha \text{ sec. } \beta + \frac{\alpha^2\ v \text{ tan. } \beta}{\text{cos. } \beta - \text{sin. } \beta\ .\ \alpha}.$$

The last term, containing α^2 as a factor, is an infinitesimal of the second order in comparison with the preceding term having α as a factor; hence, by a well-known theory of infinitesimals, the last term can be omitted in comparison with the preceding, giving, as before, $P\ Q = v\ \alpha$ sec. β.

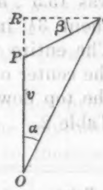

FIG. 21.

Turning now to the subject of principal stresses, the term "principal stress" in the case of beams subject to flexure, is to be understood in the usual sense as defined in the textbooks. Thus, Morley,* in addition to the usual figure, showing the directions of the principal stresses at various points in a short prismatic beam of rectangular cross-section, gives the magnitudes of the principal stresses for all points in a vertical section where the shear is large. A figure corresponding to this, but to a larger scale, is given by Turneaure and Maurer.†

In the case of the wedge-shaped beam without reinforcement, Mr. Janni's Fig. 9‡ represents the directions of the principal stresses at

* "Strength of Materials," p. 138.

† "Principles of Reinforced Concrete Construction," Art. 46.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 765.

Mr. Cain.

the various points of the cross-section approximately, about as the writer would conceive them. It was because Mr. Janni identified these intermediate "principal stresses" with the writer's "intermediate bending stresses" that he objected. In the hope that the distinction may be defined clearly, an attempt has been made to draw the principal compressive stresses at various points on the section, $I\ N$, of Fig. 22 (3), from analogy to the figures of the textbooks referred to. The principal tensile stresses on $O\ N$ are not drawn. The figure is supposed to represent part of the toe of a reinforced concrete beam, subjected to a load acting vertically upward. The upper part of the beam is supposed to make an angle, β, with the horizontal.

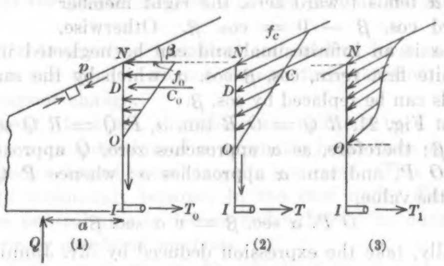

FIG. 22.

Professor O. H. Basquin,* has kindly sent the writer what he styles a "Rough Test of a Wedge-Shaped Glass Beam", with polariscope, to show the inclination of the principal compressive stresses with the horizontal at the points indicated in Table 2. The value of β was 15.5°, and the small beam was subjected to a single vertical load about $3\frac{1}{2}$ in. from the section where the measurements were taken. The entire vertical section was divided into fourteen equal parts, and the center of each (where the angle was measured) was numbered from the top down, consecutively, 1, 2, 3, . . . The results are given in Table 2.

TABLE 2.

Point.	Angle.	Point.	Angle.	Point.	Angle.	Point.	Angle.
1	15.6°	5	17.7°	9	76.4°	13	89.2°
2	14.9	6	22.2	10	83.9	14	89.5
3	14.4	7	35.9	11	86.7		
4	15.9	8	60.5	12	88.0		

"The values given for angles away from center of beam may have an error of about half a degree with a larger probable error near the center."

* Northwestern University, Evanston, Ill.

The results are very interesting. It will be observed that the angle first decreases slightly and then increases. For one-fourth of the way down, it is practically constant and nearly equal to β. Mr. Cain.

The writer's assumption as to his so-called "bending stresses" is shown in Fig. 22 (2). These stresses, for a given β, are all taken parallel to the upper surface, and increase regularly in going from the neutral axis, O, to N.

If, in Fig. 22 (3), the tensile principal stresses acting on $O\ N$ are supposed to be added, the relation between the state of stress in the revised figure (which represents all the stresses acting on the section, $I\ N$) and that given in Fig. 22 (2) can be shown by aid of the ellipse of stress in a manner kindly suggested by Professor H. A. Thomas,* in a recent letter to the writer. At a given point on $O\ N$, let t = principal tensile stress and c = principal compressive stress (at right angles to t); then the ellipse of stress can be drawn with t and c as semi-axes, Fig. 23. From this, the resultant stress, r, at the point on the plane, $I\ N$, can be found; otherwise, it can be found by Rankine's well-known construction. This resultant, r, can be decomposed into a component, p, parallel to the compressed surface, and a vertical component. A similar construction being supposed to be effected for each point of $O\ N$, the fundamental assumption made by the writer can be stated thus: the component of the resultant stress, r, parallel to the surface in compression, at any point of the reference plane, $I\ N$, is assumed to increase directly as the distance of the point from the neutral axis. The neutral axis may be defined as the line where the parallel component vanishes.

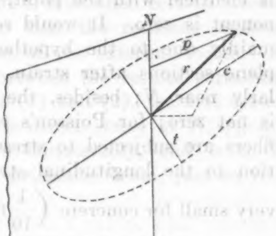

FIG. 23.

This is in agreement with the common theory of flexure for prismatic beams, which states that "the normal component of the resultant stress on the reference section, at any point, is assumed to increase directly as the distance of the point from the neutral axis".

This use by Professor Thomas of the ellipse of stress furnishes a scientific basis for the system of parallel stresses proposed, and shows very clearly that no one of the parallel stresses is a principal compressive stress, except at N, where the principal stress is parallel to the surface. Further, when this construction is supposed to be made for each point of $O\ N$, Fig. 22 (3), the result will be, not only the system of parallel forces, Fig. 22 (2), but, in addition, a vertical force acting downward along $N\ O$, equal to the sum of the vertical

* Rose Polytechnic Institute.

Mr. Cain. components referred to in connection with Fig. 23. This force, together with the sum of the vertical components of the parallel stresses, constitutes the total vertical shear on the part of the section under compression. As just shown, the parallel stress at some point, O, is zero, and if the hypothesis that plane sections, $I\ N$, remain such after strain is realized, then the original analysis shows (at least for a constant, E), that the linear law of variation holds, or the parallel stresses increase uniformly from O to N.

It thus appears that the system consisting of the uniformly varying parallel stresses of Fig. 22 (2), with the vertical force acting along $N\ O$, is the exact equivalent of the stresses shown in Fig. 22 (3), with the tensile principal stresses added. It follows that the moment of the system of Fig. 22 (2) about I (say), is exactly equal to the moment about I of the stresses of the other system. This tacitly assumes that the point, O, of Fig. 22 (2), as determined by the formula, is identical with the point, O, of System 3, where the parallel component is zero. It would seem that any error, in this particular, is mainly due to the hypotheses being inexact. The conservation of plane sections after strain, may not be sufficiently realized, particularly near N; besides, the modulus, E, is not a constant when β is not zero; for Poisson's ratio is really involved, since the interior fibers are subjected to stresses at right angles to their axes in addition to the longitudinal stresses. But Poisson's ratio is said to be very small for concrete $\left(\frac{1}{10} \text{ to } \frac{1}{8}\right)$, and the error resulting from ignoring it should be small. Of course, in place of the last two hypotheses, the formulas could all be derived from the one assumption of uniformly increasing stresses, but perhaps the results would not be so obvious.

It may be well to remark that p, in Fig. 23, is an oblique stress, per unit of area of the vertical plane, whereas the unit stress, f, of Fig. 22 (2) and of the formulas, acts normally to a plane perpendicular to the surface; therefore, $p \times 1 = f \cos. \beta$, or $f = p \sec. \beta$. Hence f varies with p and thus follows the linear law with p.

The writer will now try to make clear the term "intermediate bending stress" in connection, say, with a homogeneous toe-beam. In Fig. 23, if r is resolved into a vertical component and a component, p', the inclination of which to the horizontal at N is β, and which is then decreased according to some arbitrary law, gradually from β at N, to zero at I, as the point considered moves down the joint, the system of stresses, p', would constitute the system of "intermediate bending stresses". Below O (were $p' = 0$) the stresses would be tensile. Such a conception of "intermediate bending stresses" was entertained to try to bridge over the gap between the states of stress of reinforced and homogeneous toe-beams; it was not used, and is not necessary, in the development of the formulas, and it will be

entirely eliminated from the discussion that follows. In this, a discussion of such experiments as are at present available will be given the better to estimate the degree of security of the formulas. Mr. Cain.

From the experiments on dams, discussed later, it seems highly probable that plane sections, $I\ N$, do not remain such after strain in the vicinity of N. In fact, the resistance there, to the left of $I\ N$, is less than for a prismatic beam of the height, $I\ N$. If so, the value of f_c, Fig. 22 (2), as computed from the formula, will be in excess, or the result will be on the side of safety, as originally claimed. Direct experiments on wedge-shaped beams could easily be made to decide as to whether the conservation of plane sections after strain is sufficiently realized, also to determine the neutral axis and the values of f_c and other parallel stresses near the surface. It is the writer's belief that some of the latter are nearly as large as the computed f_c and that the actual principal compressive stress at N is less than f_c as computed. In fact, if the latter statement is true, then certain stresses below N must be largely increased, so that the resisting moment about I may equal the constant moment of the external forces. As a comparison will ultimately be made between the results pertaining to the systems of stress shown in Fig. 22 (1) and (2), the first system (1) will now be examined. In this, the direct stress at N, f_0, due to bending (using Morley's term) acts horizontally, and is to be found by the well-known formula for a prismatic beam, just as if β was zero.

$$f_0 = \frac{2\ M_c}{k_0\ j_0\ b\ d^2}.$$

This formula is found in any of the textbooks on reinforced concrete, or it is given by Equation 12 of the writer's paper,* provided k_0 and j_0 denote the values of k and j for $\beta = 0$, the steel bars being horizontal. Since the moment of the external forces about I is exactly equal to the moment of the internal stresses about I, we can put $M = M_c$. For brevity, write,

$$\frac{2\ M_c}{b\ d^2} = \frac{2\ M}{b\ d^2} = c\ ;$$

then the formula can be written,

$$f_0 = \frac{c}{k_0\ j_0}.$$

It was long ago pointed out, in the case of dams at least, that f_0 was only one conveniently chosen component of the resultant stress at N, which last acts parallel to the surface on a plane perpendicular to it. Call this principal stress p_0; then if f_0 is really the true

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 754.

Mr. Cain.

normal unit stress on a vertical plane, it follows that $p_0 = f_0 \sec.^2 \beta$.*

Now, for large values of β, with M and therefore f_0 constant, it is seen that $p_0 = f_0 \sec.^2 \beta$ becomes very large, increasing indefinitely toward infinity as β approaches 90 degrees. As this is totally unreasonable, it must follow, for a given M, b, and d, that f_0 must decrease as β increases, so that $p_0 = f_0 \sec.^2 \beta$ shall remain within reasonable limits as β increases. Also since $\sec.^2 \beta$ is a continuous function when $\beta < 90°$, increasing with β, and since $p_0 = f_0$, for $\beta = 0$, is an accepted value, it seems very probable that f_0 decreases continuously as β increases from zero, the decrease being small for small values of β and becoming greater and greater as larger values of β are taken. This probability becomes a certainty when it is considered that the toe to the left of $I\,N$, Fig. 22 (1), is less resisting than a prismatic beam of the cross-section, $I\,N$, of the same length, and consequently the actual value of f_0 at N, for the toe, is less than that for the prismatic beam of the same length. This is easily realized, especially for large values of β, by supposing the toe to be cut out of the corresponding prismatic beam, when it is apparent that near N, there is less matter to resist horizontal forces to the left than to the right of $I\,N$, and the conservation of plane sections after strain, may not apply there.

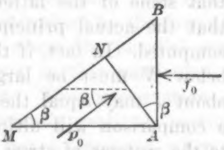

FIG. 24.

It appears, further, that the excess in the computed value of f_0, over the true value for the toe, should be very small near the truncated end of the toe, since the resistances, as regards bending of the toe and the corresponding prismatic beam (the common section, $I\,N$, being now near the end of the toe) are nearly equal. As the section, $I\,N$, Fig. 22, is taken farther from the end of the toe, the excess should increase and reach a maximum where the toe joins the stem.

The state of the horizontal stress on the vertical plane, $I\,N$, is probably as indicated in Fig. 25, the unit stress at N being less than the value of f_0 as computed by the foregoing formula, the unit stress then increasing for a little distance below N and then decreasing as O is approached, the sum of the moments of all the stresses on the cross-section about I being equal to M, as before. The unit stresses, a little below N, are thus greater than the ordinary theory would give, and plane sections, as $I\,N$, do not remain so after bending, otherwise the stresses would follow a linear law of variation from

* In Fig. 24, let $A\,N$ be at right angles to the surface, $M\,N$. Then, when $A\,N$ is regarded as infinitesimal, the stress on it is $p_0 \,.\, A\,N = p_0 \,.\, A\,B \cos.\, \beta$, and its horizontal component is $p_0 \,.\, A\,B \cos.^2 \beta$. For equilibrium of $A\,N\,B$, this must equal the horizontal stress on $A\,B = f_0 \,.\, A\,B$. On equating and solving, we find, $p_0 = \dfrac{f_0}{\cos.^2 \beta} = f_0 \sec.^2 \beta$, a formula that seems to be better known in Europe than in America.

Mr. Cain.

O to N. The same reasoning would indicate a similar modification in the state of stress shown in Fig. 22 (2), so that f_c, as computed by the formula, is in excess of the true value. It is to be hoped that direct experiments on wedge-shaped beams will soon be instituted to test these inferences. Lacking these, the experiments on india-rubber dams of Wilson and Gore and those on plasticine dams of Ottley and Brightmore* will be appealed to. From these experiments it was ascertained that the vertical unit stress on horizontal planes did not vary uniformly, the stresses near the down-stream face being generally less than corresponds to the law of uniform variation, the difference being practically nothing in the upper part of the dam and increasing as the foundation is approached, where it is comparatively large.

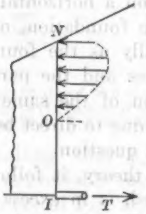

FIG. 25.

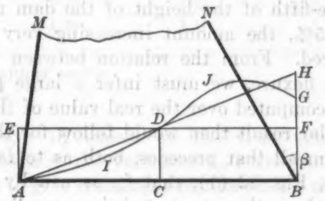

FIG. 26.

The variation of this stress is shown in Fig. 26, which may be taken as typical of the results of the various experiments. In this figure, $A\ B$ is a horizontal section of the dam, $A\ B\ N\ M$, near its base, the angle, β, being 40° 55′ for the plasticine dam and 32° for the straight portion of the rubber dam. The resultant on the section was supposed to cut it, $\frac{1}{3}\ A\ B$ from B, so that the vertical ordinates from AB to a straight line, $A\ H$, represent the stresses in question, according to the usual theory. Also, if C is the mid-point of $A\ B$, the vertical ordinates from $A\ B$ to $E\ F$ represent the uniform compression due to the supposed vertical load at C. The ordinates from $D\ F$ to $D\ H$ and from $D\ E$ to $D\ A$ represent the stresses due to the bending moment, all according to the ordinary theory. By experiment, the vertical unit stress on $A\ B$, at any point of it, is given by the vertical ordinate there, from $A\ B$ to the curved line, $A\ I\ D\ J\ G$. The variation of stress due to a central load at C is unknown, but it is certainly not uniform, and the stress is a little less than $B\ F$ at B, and probably a little greater than $C\ D$ at C, the variation being given by the ordinates from $A\ B$ to a slightly curved line between the extreme points, and not by $E\ D\ F$. The actual unit stresses corre-

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907–08, Part II, see Pl. 3, Fig. 4, and Pl. 5, Figs. 19–21.

Mr. Cain. sponding to the bending moment will be represented by the vertical ordinates between this slightly curved line and $A\ I\ D\ J\ G$. This (for the part, $C\ B$, of the section) would give about the variation of stress shown in Fig. 25. The unit stress, corresponding to the bending moment, that particularly interests us, is that at B, Fig. 26, which is represented by a vertical line from G to a point slightly below F. This stress will be smaller than $F\ H$, and, similarly, we should conclude for the toe, Fig. 22 (1), that f_0 as computed is too large (at least for $\beta = 32°$ to $41°$), though the excess over the true value is negligible near the truncated end of the toe and increases as the stem is approached, where it is comparatively large.

No quantitative results can well be given, but a glance at the diagrams relating to the experiments on the dams referred to will show an excess in the total vertical unit stress on a horizontal plane, say one-fifth of the height of the dam above the foundation, of more than 25%, the amount increasing very materially as the foundation is neared. From the relation between this stress and the part of it due to flexure, we must infer a large proportion of the same excess in the computed over the real value of the stress due to direct bending. A similar result then would follow for the toe in question.

From all that precedes, both as to facts and theory, it follows, for the toe, Fig. 22 (1), that f_0, as usually computed, is in excess of the true value, the amount being small for small values of β and increasing with β, slowly at first and more rapidly when β is large, the excess being enormous when β approaches 90 degrees. Further, it would appear that this excess should be very small when the section, $I\ N$, is taken near the truncated end of the toe, and that it should increase as the vertical section is taken nearer its junction with the stem of the retaining wall, where it is a maximum.

A comparison will now be made between the results obtained in computing the principal stress at N, (1) by the formula previously given,

$$p_0 = f_0 \sec.^2 \beta = \frac{c}{k_0 j_0} \sec.^2 \beta$$

and (2) by use of Equation 15 of the writer's paper,*

$$f_c = \frac{2\,M}{kj\,bd^2 \cos.^2 \beta} = \frac{c}{k\,j} \sec.^2 \beta,$$

where, $c = \dfrac{2\,M}{bd^2}$, Fig. 22 (2), showing the assumed distribution of stresses along the vertical section.

On dividing the first equation by the second, we have,

$$\frac{p_0}{f_c} = \frac{k\,j}{k_0\,j_0}$$

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 756.

where k, j, refer to System (2) of stresses, the values being taken from the full lines of the diagram, Fig. 6, of the writer's paper,* for the assumed value of β. The values of k_0, j_0, may be found on the same diagram from the line corresponding to $\beta = 0$. Mr. Cain.

The results are given in Table 3 for various values of β and $p = \frac{A}{b\,d}$.

TABLE 3.—STEEL PERCENTAGE, 100 p.

$100\,p =$	0.2	0.6	1.0	2.0
	$\frac{p_0}{f_c}$	$\frac{p_0}{f_c}$	$\frac{p_0}{f_c}$	$\frac{p_0}{f_c}$
$\beta = 10°$	1.02	1.02	1.01	1.00
20°	1.05	1.05	1.04	1.02
30°	1.13	1.11	1.09	1.06
40°	1.24	1.20	1.17	1.12

In Fig. 22, $d = I\,N$, b = constant width of section.
In Fig. 22 (1), $k_0\,d = N\,O$, $j_0\,d = D\,I$.
In Fig. 22 (2), $k\,d = N\,O$, $j\,d = D\,I$.

Recalling that p_0 is always in excess of the true value, the excess appreciably increasing with β, the values of f_c, corresponding to System (2) of Fig. 22, are seen to give consistent results and in the right direction. The regularity of the changes in the ratio is noticeable, and the near approach to unity for $\beta = 10$ to 20 degrees. The excess of p_0 over f_c, which varies from 0 to an extreme of 24% for $100\,p = 0.2$, $\beta = 40°$, is what might have been expected from the diagrams relating to the experiments. Also, since the computed value of p_0 for the toe, is in excess (much more than 25%) near or at its junction with the stem for $\beta = 32°$ or $\beta = 41°$, reasoning from the results of the experiments on dams, it is seen from Table 3 that f_c for $\beta = 32°$ or $\beta = 41°$, is in excess of the true value, and, presumably, f_c is in excess for any value of β considered.

It is impossible to draw certain conclusions when experiments in great variety are lacking, but all the results, theoretical and experimental, given thus far tend to support the approximate formula for f_c as a good practical formula within the limits $\beta = 0$ to $\beta = 45°$, and on the side of safety. Further, the value of f_c, as just computed, is much nearer the truth than the value of $p_0 = f_0 \sec.^2 \beta$.

It has been suggested that perhaps $f_0 \sec. \beta$ would be preferable to use, as a semi-empirical formula, in place of $f_0 \sec.^2 \beta$, which gives results too large. Calling $p_0' = f_0 \sec. \beta$, the ratio $\frac{p_0'}{f_c}$ was made out for different steel percentages and values of β, as given in Table 4.

* Loc. cit., p. 755.

Mr. Cain.

TABLE 4.—STEEL PERCENTAGE, 100 p.

$100\,p =$	0.2	0.6	1.0	2.0
	$\frac{p'_0}{f_c}$	$\frac{p'_0}{f_c}$	$\frac{p'_0}{f_c}$	$\frac{p'_0}{f_c}$
$\beta = 10°$	1.01	1.01	1.00	0.99
20°	0.99	0.99	0.98	0.96
30°	0.98	0.97	0.95	0.92
40°	0.95	0.92	0.90	0.86

As p_0' is generally less than f_c, until more decisive experimental evidence is forthcoming, it seems unwise to use the smaller value.

Although the previous investigation shows that the value, p_0, is always in excess, and that f_c is nearer the true value, the question at once occurs: why not use, for such an uncertain quantity as concrete, the larger value?

By reference to Table 3, it is seen that p_0 exceeds f_c by from 1 to 5% only, for $10° < \beta < 20°$ and by from 6 to 24% for $30° < \beta < 40°$, for the various steel percentages. The average excess is small; besides, the junction of toe and stem is a recognized point of weakness in concrete construction, so that a decided excess value seems desirable. Further, the method of computation of p_0 is simple and more generally understood than the method by which f_c is computed, which introduces the novel hypothesis of parallel stresses. If this method is adopted in the computation of f_0, Fig. 22 (1), the values of k_0 and j_0 can be found from usual diagrams pertaining to prismatic beams, the principal stress in the concrete computed from the formula, $p_0 = f_0 \sec.^2 \beta$, and the stress in the steel (which is usually the determining factor) found as usual. If p_0 is assigned a limit of say, 650 lb. per sq. in., then $f_0 = 650 \times \cos.^2 \beta$; with this value and the maximum unit stress allowed in the steel, the usual French diagram can be consulted. The serious drawback to the use of this method is that f_0 is known to be in excess, and that the state of stress shown in Fig. 22 (1), cannot be justified. We have seen, though, that the state of stress shown in Fig. 22 (2) is on a more satisfactory basis, in that it introduces β and f_c from the start, and that it may be near the true state, any variation being most marked near N. At any rate, it is a good working hypothesis, consistent with the few experiments available, and to be tested by future more direct experiments. The formulas corresponding to the hypotheses have been developed by rigorous methods, and the resulting values of f_c agree fairly well with the results of the experiments quoted. It seems to be common practice (which Mr. Janni endorses,

when "β is not large") to use f_0 as the "maximum compressive stress" in the concrete, in place of either p_0 or f_c. It would appear from the foregoing investigation that such practice may be considerably on the danger side, and should be condemned—certainly until more decisive results can be reached from future experiments. Mr. Cain.

The previous discussion deals with the case where $\beta > 0$ and $\beta_1 = 0$. For the case of the heels and counterforts of retaining walls, $\beta = 0$, and, generally, some of the steel bars are not at right angles to the section, $I\ N$. For this case, all the general formulas of the writer's paper hold on putting $\beta = 0$. This is a much simpler case than the former, since the only hypothesis introduced is that plane sections remain such after bending.

In the writer's first draft of his paper, this case was considered by itself, and, following it, the case where $\beta > 0$, which introduced a second hypothesis as to parallel stresses. Both cases were then combined into one to save space in the final writing. It is regretted now that the two cases were not considered separately, as then the merits or demerits of each could have been more easily seen or discussed.

Taking up again Mr. Janni's further comments, it may be observed that his Fig. 17 shows a buttressed wall, which is not the case of the counterforted wall investigated by the writer. In the latter, the earth is to the right of the face wall, the counterfort being buried in it. The force, F, of Fig. 17, thus acts to the left. In practice, too, the friction forces are usually neglected, for simplicity. The base of the counterfort is connected to the foundation slab (which is loaded with earth) by steel rods, vertical and inclined, so that the counterfort can be regarded as a beam fixed at the ends or at its junction with the base slab. It is true that it is a very short beam, so that the effect of shear is very large; still, a bending moment has to be accounted for, and beam action is thus induced. The theory of such beam action enables one to compute the stresses in the rods, as was illustrated in connection with Fig. 3.*

With regard to the toe-beam, Fig. 18, if the friction is included, the "principal stress" at P' is, of course, inclined to the horizontal; but the writer had in mind the "bending stress", as explained in the foregoing, which was taken as acting parallel to $L\ I$, according to the original assumption. Mr. Janni is right in his remark about the modulus, E, not exactly applying, as before remarked. It was considered, however, that in an approximate computation, the Poisson ratio consideration could be ignored, particularly as in concrete this ratio is very small. In all approximate computations, one or more assumptions, not exactly true, are usually made, it being considered that the degree of accuracy in the final computations is the final test, and, from what has preceded (see Table 4) the computations for f_c

* *Loc. cit.*, p. 751.

Mr. Cain.

stand the test for practical accuracy remarkably well. The ordinary modulus could evidently be used near the surface, where the larger stresses with the larger arms are most effective in resisting the bending couple, so that the error should not be large in using a constant modulus throughout.

As to shear, let Fig. 27 represent an infinitesimal rhombic prism at the neutral axis, two faces being parallel to the neutral surface, *OO′*, and the other faces being vertical. The stress on the two vertical faces shown is a pure shear, *s*. As there is generally a vertical component on all planes parallel to *OO′* to the compressed surface, the possibility of such a component must be recognized at the neutral surface. Then, if we resolve *r*, the resultant unit stress on *O O′*, into a vertical component, *t*, and one acting parallel to *O O′*, it follows that the latter is equal to *s*, as is shown by taking moments of the forces shown, say about *O*. By reference to Fig. 8,* it is seen that *s*, this component of *r*, is given by Equation 17, and is thus identical in value with *v* of the formula. The unit shear, *s*, on a vertical plane is likewise equal to the unit shear on a horizontal plane at *O*, from the well-known theorem. In the demonstration, *OO′* was assumed to be parallel to the upper surface, which is not strictly true; in addition, *V* should have the meaning given later, where shear is thoroughly discussed. It can be easily shown, by a slight modification of usual methods, that the unit shear component, *s* (as just defined), on a plane parallel to *OO′* (assumed to be parallel to the surface) and at a vertical distance, *y*, above it, is given by the formula,

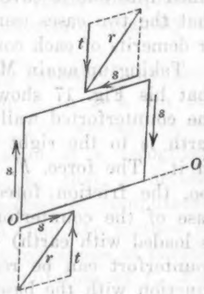

Fig. 27.

$$s = \frac{V}{b\,(k\,d_1)^2 \cdot D\,G}\,[(k\,d_1)^2 - y_1^2]^2.$$

Thus, this component, *s*, varies according to the parabolic law, and is zero when $y_1 = k\,d_1$ or at the surface, as we know to be true from the elementary considerations given in connection with Fig. 1.†

This component, *s*, is thus exactly the shear on a horizontal or vertical plane at *O*, and "above the neutral surface, it decreases, by the usual parabolic law, to zero at *N*". It is not the shear on a horizontal or vertical plane between *O* and *N*. That will be given later.

It will be observed, at the point, *O*, that if *r*, Fig. 27, is resolved into components perpendicular and parallel to *OO′*, the latter—the full shear on *OO′*—may be greater than *s*. The vertical component was

* *Loc. cit.*, p. 758.

† *Loc. cit.*, p. 746.

overlooked in the writer's first analysis. The decomposition indicates that the true shear on OO' may be greater than s. Mr. Cain.

Mr. Janni again introduces Fig. 10,* in which it seems that he intends to leave the part of the beam to the left of $I\ N$ unchanged, as MN shifts to the different positions shown by the dotted lines. As β approaches 90°, the value of f_c, given by him, becomes smaller and smaller for a constant M, and at $\beta = 90°$, $f_c = 0$, which is characterized as absurd. Messrs. Slocum and Hancock† give a complete solution of this case. These authors conclude that "if a piece is bent exactly at right angles on itself, the slightest bending strain must produce incipient rupture"; also, that "the neutral axis passes through the vertex of the angle" (N in Fig. 10). This seems to dispose of Mr. Janni's "absurd conclusion".

As to the solution proposed by Mr. Janni in connection with Fig. 19, no objection can be urged, provided the inclination of the reinforcing bar to the section, $N\ I$, is properly included in deducing the formulas for k, etc. If this is not done, then the solution would be "flatly wrong"; but, supposing a correct solution to have been made, the resulting computations would be long and wearisome, as note the corresponding solutions for a prismatic beam, subjected to "flexure and direct stress" given by Turneaure and Maurer.‡ The solution given by the writer,§ in connection with the diagram for finding k and j, is very short and practical.

The concluding remarks of Mr. Janni about the neutral axis being fixed, if the hypothesis of parallel stresses is adopted, is inexplicable to the writer; for this hypothesis leads by rigorous mechanical laws to a value of k which varies with β, β_1, etc. It may be added, likewise, that whatever objection applies to the determination of k, with regard to the state of stress shown in Fig. 22 (2), would seem to apply equally to the state of stress shown in Fig. 22 (1), when β was not zero. The exact determination of O is doubtless a very complicated problem, which awaits a solution. The writer's only aim was to get an approximate result leading to safe formulas.

The writer has studied more carefully the subject of shear, since the publication of his paper, and is glad of this opportunity to state his agreement, in part, with Mr. Nishkian in his contention that the vertical component of the stress in the inclined rod should be deducted from the vertical load to get the shear in the concrete of the heel, Fig. 12.‖ Thus, to take a more general case, consider the toe, Fig. 28, subjected to the vertical load, Q. The stress, C, inclined at the angle, β,

* *Loc. cit.*, p. 766.
† "Text-book on the Strength of Materials," Art. 127.
‡ "Principles of Reinforced Concrete," Arts. 80–85.
§ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 756.
‖ *Loc. cit.*, p. 768.

Mr. Cain. the tension in the rod at I inclined at the angle, β_0, to the horizontal, and the shear, V, in the section, $N\,I$, to be carried by the concrete, in addition to the vertical component of C already included. V is thus the shear in the section still to be accounted for after the vertical components of C and T have both been considered. For equilibrium, we must have,

$$V = Q - C \sin. \beta - T \sin. \beta_0.$$

This portion, $L\,M\,N\,I$, of the beam acts on the part of the beam to the right of the section, $N\,I$, with forces exactly equal and opposed to C, T, and V.

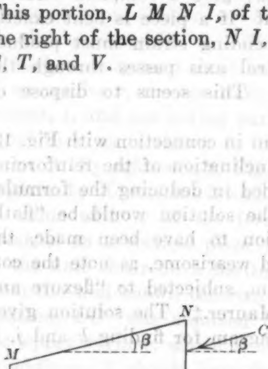

FIG. 28.

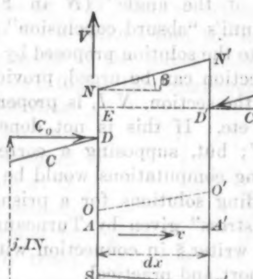

FIG. 29.

When the steel is horizontal, $\beta_0 = 0$ and $V = Q - C \sin. \beta$. This refers to the case shown in Fig. 22 (2). For the case shown in Fig. 22 (1), C is horizontal and $V = Q$. When T, Fig. 28, acts upward, the last term in the first formula changes sign. In the first investigation of shear, C' was assumed to act in the same line as C, which is only approximately true. To make an exact analysis, consider Fig. 29, representing a portion of the beam between the two vertical sections, $I\,N$ and $I'\,N'$, dx apart and subjected to the forces, V, V', C, and C' (the horizontal components of which are C_0, C_0'), T, the resultant of the tensions in the bars, $I\,I'$, $L\,L'$, $S\,S'$, acting at G and inclined at the angle, β_0, to the horizontal and the corresponding resultant, T', at G', supposed to act in the line of T, but opposite in direction. The constant width of the cross-sections will be called b, and the unit shear

on a horizontal plane, $A\ A'$, just below the neutral surface, v. We have, Mr. Cain.

$$I'\ D' - I\ D = j\ (I'\ N' - I\ N) = j\ dx\ (\tan.\ \beta + \tan.\ \beta_1)$$

$$D\ E = I\ E - I\ D = I'\ D' - dx\ \tan.\ \beta_1 - I\ D = j\ dx\ (\tan.\ \beta + \tan.\ \beta_1) - dx\ \tan.\ \beta_1.$$

Now, whatever the inclinations assumed of C and C', we have

$$C_0 = T\ \cos.\ \beta_0 \qquad (24)$$

$$C_0' - C_0 = v\ b\ dx \qquad (25)$$

the latter equation resulting from placing the sum of the horizontal components of the forces acting on $A\ A'\ N'\ N$ equal to zero. Two cases will be considered, corresponding to the dispositions of stress in the concrete shown in Figs. 22 (1) and (2).

In the first case, C and C' are taken as horizontal; whence $C = C_0$. $C' = C_0'$. The corresponding value of j (call it j_0) can be derived from the proper formula (or diagram, for one layer of rods at $I\ I'$) on taking $\beta = 0$.

On taking moments of the forces acting on $I\ I'\ N'\ N$ about G, we have,

$$C_0'(G\ D + D\ E) - C_0\ .\ G\ D = V'\ dx \qquad (26)$$

therefore $\quad (C_0' - C_0)\ G\ D = V'\ dx - C_0'\ .\ D\ E;$

whence, by equations above,

$$v\ b\ dx\ .\ G\ D = V'\ dx - C_0'\ [j_0\ (\tan.\ \beta + \tan.\ \beta_1) - \tan.\ \beta_1]\ dx.$$

Divide by dx and take the limit as dx tends toward zero; so that V' and C_0' are replaced by $V = (Q - T\ \sin.\ \beta_0)$ and C_0.

$$v\ b\ .\ G\ D = Q - T\ \sin.\ \beta_0 + C_0\ \tan.\ \beta_1 - j_0\ C_0\ (\tan.\ \beta + \tan.\ \beta_1)\ .\ (27)$$

The usual case is where there is only one layer of bars, $I\ I'$; whence $\beta_0 = \beta_1$, $G\ D = I\ D$, and, by Equation 24, $C_0\ \tan.\ \beta_1 = T\ \sin.\ \beta_1 = T\ \sin.\ \beta_0$. Therefore,

$$v\ b\ .\ G\ D = Q - j_0\ (C_0\ \tan.\ \beta + T\ \sin.\ \beta_1) \qquad (28)$$

where $G\ D = j_0\ .\ I\ N$.

In the second case, C and C' will be assumed to act parallel to $N\ N'$. Therefore, $C_0 = C\ \cos.\ \beta$, $C_0' = C'\ \cos.\ \beta$, and j is to be found as usual, for the given value of β. The right member of Equation 26 is now to be increased by $C'\ \sin.\ \beta\ .\ dx$; whence, proceeding as before, and substituting, $V = Q - T\ \sin.\ \beta_0 - C\ \sin.\ \beta$, as just explained, we have,

$$v\ b\ .\ G\ D = Q - T\ \sin.\ \beta_0 + C_0\ \tan.\ \beta_1 - j\ C_0\ (\tan.\ \beta + \beta_1)\ .\ (29)$$

where $C_0 = C\ \cos.\ \beta$. For one layer of bars $I\ I'$, $\beta_0 = \beta_1$, $G\ D = I\ D$, $C_0\ \tan.\ \beta_1 = T\ \sin.\ \beta_0$. Therefore

$$v\ b\ .\ G\ D = Q - j\ (C\ \sin.\ \beta + T\ \sin.\ \beta_1) \qquad (30)$$

where $G\ D = j\ .\ I\ N$.

Mr. Cain.

In Equations 27 to 30, on dividing by $b \,.\, G\,D$, we find the shear, v.

Note, in the right member of Equation 30, that in place of subtracting the sum of the vertical components of C and T from Q, that only j times this amount is subtracted.*

The bond stress for the general case of many layers of bars, Fig. 29, is easily found. Using the previous notation,† and since the shear on the horizontal plane, $A\,A'$, equals the sum of the horizontal components of the bond stresses in the bars, Fig. 27,

$$b\,v\,dx = u_1\,I\,I' \cos.\,\beta_1\,\Sigma o_1 + u_2\,L\,L' \cos.\,\beta_2\,\Sigma o_2 + \ldots$$
$$= dx\,(u_1\,\Sigma o_1 + u_2\,\Sigma o_2 + \ldots)$$

Therefore, $$b\,v = u_1 \left[\Sigma o_1 + \frac{O\,L}{O\,I}\,\Sigma o_2 + \frac{O\,S}{O\,I}\,\Sigma o_3 + \ \ldots \right] \ldots\ldots (31)$$

For any one of the cases corresponding to Equations 27 to 30, substitute the value of $b\,v$ corresponding, in the foregoing equation, whence u_1, and, subsequently, $u_2 = u_1\,\dfrac{O\,L}{O\,I}$, etc., can be found. For only one layer of bars, $u_1 = \dfrac{b\,v}{\Sigma o_1}$, and using, say, Equation 30, also placing $D\,G = D\,I = j\,.\,I\,N$,

$$u_1 = \frac{Q - j\,(C \sin.\,\beta + T \sin.\,\beta_1)}{j\,.\,I\,N\,\Sigma o_1} \ldots\ldots\ldots\ldots\ldots (32)$$

The formulas for shear and bond stress just given are to replace the corresponding formulas for shear and bond stress hitherto given. In the applications, it may prove desirable to express C_0 in the "first case" by $T \cos.\,\beta_0$, and in the "second case" to place $C = T\,\dfrac{\cos\,\beta_0}{\cos.\,\beta}$. In all the formulas, Q represents the algebraic sum of the reaction (if any) and the loads left of the section $I\,N$. Also, if T' is directed to the right and upward, $\sin.\,\beta_0$ is changed to $(-\sin.\,\beta_0)$. Similarly, change β_1 to $(-\beta_1)$ if $I\,I'$ lies above the horizontal through I, with a consequent change of sign of $\sin.\,\beta_1$ and $\tan.\,\beta_1$.

It will be observed that, in consequence of the term subtracted from Q being often large, the values of v and u are small as compared

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* In Mörsch's "Concrete-Steel Construction" (Goodrich's translation), p. 190, similar results, for some of the simplest cases, are found by a very neat method involving the calculus. This method can easily be extended to other cases, but it is perhaps not so obvious as the above. The moment, M, of the external forces about either G or D is constant; hence, for one layer of rods, $I\,I'$, with G at I, $M = C_0\,.\,j_0\,I\,N$ in Equation 28 $= C \cos.\,\beta\,.\,j\,I\,N$ in Equation 30; therefore $j_0\,C_0 \tan.\,\beta$ in Equation 28 $= j\,C \sin.\,\beta$ in Equation 30. Similarly, if T_0 is put for T in Equation 28 to distinguish it from the T in Equation 30, $T_0\,j_0\,I\,N$ in Equation 28 $= T\,j\,I\,N$ in Equation 30; therefore, $T_0\,j_0 \sin.\,\beta_1$ in Equation 28 $= T\,j \sin.\,\beta_1$ in Equation 30. Whence the right members of Equations 28 and 30 are equal in value; so that the values of v derived from Equations 28 and 30 are in the ratio of the $G\,D = j\,.\,I\,N$ of Equation 30 to the $G\,D = j_0\,.\,I\,N$ of Equation 28, or of j to j_0. The values j and j_0 differ by only a small percentage.

† *Loc. cit.*, p. 760.

with results obtained for prismatic beams where $\beta = 0, \beta_1 = 0$. Hence it will rarely be necessary to use the formulas above in their entirety, since a rough estimation of the subtractive term will generally suffice to indicate safe limits. Mr. Cain.

It has been remarked, in connection with Fig. 27, that the unit shear at *O*, on a plane inclined to the horizontal, may be greater than the shear on the horizontal plane. The difference will usually be small. The unit shear, q, on a horizontal plane at the distance, y_1, above O in the case of the toe, Fig. 22 (1), for the state of stress shown in that figure, can be found by a method similar to that used for prismatic beams, only the rise of N' above N and of O' above O, Fig. 29, must be considered in the limits of the integrations, etc. The results are to be written only to first order infinitesimals, but even then, the work is long, so that only the final result will be given. As before, b = constant breadth and d = depth, $N\,I$, Fig. 22 (1), M = moment of external forces about I, and k_0, j_0, refer to the state of stress shown in the figure. The value of q is given by the following equation,

$$q\,(k_0^2\,j_0\,b\,d^3) =$$

$$k_0^2\,d\,(Q\,d - M \tan.\,\beta) + [2\,(1 - k_0)\,M \tan.\,\beta]\,y_1$$

$$+ \left[\frac{3\,M \tan.\,\beta}{d} - Q\right] y_1^2 \cdots\cdots\cdots\cdots (33)$$

The unit shear at O is found by putting $y_1 = 0$ and substituting $C_0\,j_0\,d$ for M,

$$q\,(\text{at } O) = \frac{Q - j_0\,C_0 \tan.\,\beta}{b\,.\,j_0\,d},$$

as found in another manner above. At N, the shear is found by putting $y_1 = k_0\,d$. Therefore,

$$q\,(\text{at } N) = \frac{2\,M \tan.\,\beta}{k_0\,j_0\,b\,d^2} = f_0 \tan.\,\beta,$$

as can be proved independently. Here, f_0 indicates the direct bending stress at N, Fig. 22 (1).

Another check is afforded by taking $\beta = 0$, whence,

$$q = \frac{Q\,(k_0^2\,d^2 - y_1^2)}{k_0^2\,j_0\,b\,d^3}$$

which is very easily proved independently.

Reasons have been given for the computed value of f_0 being in excess, the state of horizontal stress on the vertical plane, $I\,N$, being nearer that shown in Fig. 25 than the one given in Fig. 22 (1); hence q at $N < f_0 \tan.\,\beta$, and, similarly, the variation in q is not exactly that given by the equation, as y_1 increases from zero to $O\,N$. How-

Mr. Cain. ever, accepting it as showing roughly the variation in q, it is seen that Equation 33 can be written in the form,

$$q = l + m\, y_1 + n\, y_1^2,$$

where q and y_1 are the only variables. When $n = 0$, this is the equation of a straight line. When n is not zero, it may be positive or negative, and the equation is that of a parabola, the axis of which is horizontal, and the vertex is at the point, the abscissa of which (parallel to axis of q) is $\left(l - \frac{m^2}{4\,n}\right)$ and the ordinate (parallel to axis of y_1) is $\left(-\frac{m}{n}\right)$.

Figs. 30 and 31 show the loci for $n > 0$, $n < 0$, respectively, for assumed values, the value of q at $y_1 = O\,N$, being f_0 tan. β in both cases. In Fig. 30, q increases with y_1 from O to N; in Fig. 31, q decreases with y_1 above the vertex. Of course, only the portions of the curves from $y_1 = 0$ to $y_1 = O\,N$ pertain to the problem. If the load, Q, acts a units to the left of $I\,N$, Fig. 22 (1), we have, $M = Q\,a$, hence,

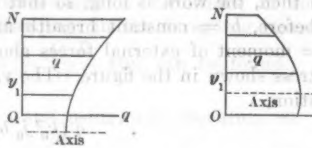

FIG. 30. FIG 31.

$$l = \frac{Q\,(d - a \tan.\,\beta)}{j_0\, b\, d^2}$$

is always positive, since $d > a$ tan. β. In fact, when $y_1 = 0$, $q = l$, or the shear at the origin acts to the right, so that the locus, whether a parabola or a straight line, cuts the axis of q to the right of O. It is evident from the general equation above that m is always positive. Also, since

$$n = \frac{Q}{k_0^{\,2}\, j_0\, b\, d^3}\left(\frac{3\,a \tan.\,\beta}{d} - 1\right),$$

it is evident that,

$$n > 0, = 0, < 0,$$

according as,

$$a \tan.\,\beta > \frac{d}{3}, = \frac{d}{3}, < \frac{d}{3},$$

corresponding, respectively, to a long toe, a medium toe, or a very short toe. When n is positive, the abscissa of the vertex may be positive, as shown in Fig. 30, or it may be negative. When n approaches zero, the vertex recedes indefinitely to the left and downward, and the locus approaches the straight line, $q = l + m\,y_1$, where m and l are always positive.

Similarly, when n is negative, Fig. 31, although the vertex of the parabola is always in the first quadrant, yet as n tends toward zero, it recedes indefinitely from the origin, and the shear increases in going from O to N, and the locus approaches again the straight line, $q = l + m\ y_1$. Mr. Cain.

The foregoing general formula only gives the unit shear, q, on a horizontal (or vertical) plane at the distance, y_1, above O. At this point, in addition to these shears, there is a horizontal pressure, f, on a vertical plane and a vertical pressure on a horizontal plane. The latter is unknown, except at the surface, $M\ N$, where it is $f_0 \tan.^2 \beta$, as can easily be proved by aid of an infinitesimal wedge like $A\ M\ B$, Fig. 23.

If the state of stress indicated by Fig. 22 (2) is assumed, the same general formula, as given above, is derived, except that k_0, j_0, are to be replaced by k, j.

The writer has now given formulas for the two states of stress indicated in both Fig. 22 (1) and 22 (2), referring to stresses due to flexure as well as to bond and shear stresses, and certain results have been tested by such experiments as were available. The case must now rest until the results of direct tests on wedge-shaped beams are available. There is a prospect that such tests will be made at several institutions within the next year.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1324

THE DIFFERENTIAL SURGE TANK*

By Raymond D. Johnson, Esq.

With Discussion by Messrs. Roy Taylor and D. L. Webster.

Synopsis.

It is believed that a good practical working knowledge of the surge tank problem is a valuable asset to the modern hydraulic engineer. The writer has gathered together, in the past twelve years, many useful formulas, has made a thorough study of the subject, and takes this opportunity of presenting a concise analysis to his fellow engineers for their consideration and criticism. Mr. Roy Taylor, the writer's assistant, has been sufficiently interested to give up months of his time patiently to work out illustrative examples, and his criticism should be carefully read as a necessary adjunct to this paper.†

The ordinary simple surge tank, or storage reservoir, which is commonly placed at the end of a long pipe line, has been entirely neglected in this paper, for four reasons which experience has demonstrated:

First.—It never costs less than the differential type for approximately the same benefit realized.

Second.—It can seldom be made quite so effective as the latter type, even at a greatly increased cost.

Third.—It is not capable, to anything like the same degree, of a rigid mathematical treatment.

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* Presented at the meeting of December 2d, 1914.

† For Mr. Taylor's discussion, see p. 785.

Fourth.—It is subject to unstable conditions of the vibrating water column, to an extent which renders an exact prediction of its behavior, if not impossible, at least far more difficult than that of the differential type.

The differential surge tank consists of a stand-pipe of about the same diameter as the conduit, freely connected to it, and a storage tank of larger dimensions, surrounding the stand-pipe and connected to the conduit by a properly restricted passage.

In a simple tank, the level of the stored water, following a demand for more power, represents the accelerating level which is urging more water from the forebay, and measures the head acting on the water-wheel. In the differential type, owing to the resistance interposed between tank and conduit, the level of the stored water is quite independent of the acceleration, and does not affect the water-wheel governor directly. The water in the stand-pipe takes care of these things, and acts like a simple tank of small dimension which is supplemented by the steadying action of the stored water, fed into the system in an independent, non-synchronous manner, meeting all demands for water without causing the unstable, pendulum-like behavior which is so characteristic of the simple surge tank.

On account of the possibility of a rigid mathematical treatment of the ideal or theoretical differential action, several interesting facts are discoverable which would probably not have been generally demonstrable by a study of the so-called simple tank, which, by the way, is anything but simple in all respects save as to the conception of its physical structure. Some of the features most noteworthy, are:

a.—The working volume of a differential surge tank, for a given velocity change and for a given degree of stability in regulation, is independent of the length of the conduit, the area decreasing as the length of conduit, and consequent height of tank, increase.

b.—The highest possible surge above forebay level following a complete shut-down, is not necessarily due to the shutting off of the maximum velocity obtainable, but may occur as a result of throwing off part load at a critical velocity.

c.—It is possible, and often quite feasible, to design for a perfect dead-beat or non-oscillatory condition, where no unnecessary

motion of the water occurs, nothing more than a change of level corresponding to the inevitable gradient change.

d.—Every design is intimately associated with an infinite number of similar designs, so related, kinematically, that the one type may be varied indefinitely in its own class, without further computation, except of a primitive nature. This fact favors graphical methods, and enables all computations to be reduced to the utmost simplicity.

These features are here brought out, along with a general mathematical treatment deduced from fundamental principles of mechanics, and the illustrations by Mr. Taylor complete the demonstration of the differential principle, and show its advantages over the ordinary method now in somewhat general use.

INTRODUCTION.

The function of a stand-pipe or simple surge tank, to regulate a long enclosed column of water feeding water-wheels, is well enough understood, and many authors have written on this subject since the presentation of the writer's paper entitled "The Surge Tank in Water Power Plants."* Therefore he will pass over the elementary matter, involving description of the undulating water wave produced by load changes, and take up the more technical side of the question, illustrating some exact mathematical methods of arriving at intelligent results in design. In the following studies the exact determinations are confined to the first quarter cycle of the water wave, that is to say, the period during which the first drop or rise of water within the tank takes place, up to the time when a reversal of motion of the water begins.

The differential surge tank or regulator, as has been described in the writer's previous paper, consists of a simple stand-pipe freely connected to the water-power conduit, and supplemented by a tank surrounding the stand-pipe and communicating with the conduit through a definitely restricted opening. This restricted passage offers a resistance to the flow of water back and forth through it, and, when properly designed and proportioned, has a remarkably beneficial effect on surges which would otherwise attain harmful, if not dangerous, proportions in a simple surge tank of much larger dimensions.

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* *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, p. 443 (1908).

No one has yet succeeded in writing correct equations for the phenomena during the first quarter cycle in a simple surge tank, when friction, velocity head, and governor action are considered. Neither is it possible to do this for the differential regulator, except for the case when the stand-pipe area approaches zero for a surrounding tank of a given size. A certain capacity of stand-pipe is essential for many reasons, as will appear later; one of its principal benefits is to furnish a little time for the pressure change following a load change, so as not to interfere with the governor and throw additional duty on the fly-wheel. It is highly instructive and very satisfactory to be able to write perfectly exact equations for all the phenomena which take place during the first quarter cycle, taking into account the friction in the conduit and eliminating without error the action of the governor. This can easily be done when the capacity of the internal stand-pipe approaches zero, and the subsequent addition of this capacity and the further refinement of the area of the restricted passage for an actual design become an easy task for a little judgment assisted by arithmetic integration.

An undulation or water wave once set in motion by a load change may have any one of four characteristics, depending on the size of the tank, other conditions being fixed.

(1) It may die out as a damped harmonic;

(2) It may live indefinitely at the same amplitude;

(3) It may increase in amplitude with each oscillation;

(4) It may continue to drop, theoretically, without turning around to rise again, if the acceleration of the conduit is not fast enough to catch up to the increasing draft velocity as the head recedes and the governor opens up the water-wheel wider and wider.

(This last case is interesting only from a mathematical standpoint, and will not here be further considered.)

(5) As a modification of Case (1), it may die out with no oscillation at the end of the first quarter cycle in infinite time. This case is called the non-oscillatory or dead-beat condition, where perfect or absolute damping takes place, and the water does not pass beyond the normal quiescent level due to the velocity required by the suddenly applied load.

When load changes happen to occur in step with the natural period of vibration of the water column, Case (1) usually takes on the characteristics of Case (3), and experience has shown that this test, applied to many simple tank designs, disqualifies them for safe use. On the other hand, it is extremely rare to hit upon a case where a differential regulator will not hold the water within definite limits up and down, however long synchronous load changes be applied. Under these conditions the vibration may sometimes increase for one or two swings, after which it will remain at constant amplitude for an indefinite number of continued synchronous load changes. The writer recently ran across a practical case where a simple tank of certain dimensions took on the characteristics of Case (3), following a single load change, and the same tank, converted into a suitable differential regulator, successfully withstood the test, even, of synchronous load changes. Case (5) is in most cases economically out of the question to obtain with a simple tank, but frequently for long pipe lines, large friction, or high velocities, or all of these, it becomes quite possible to design for the dead-beat condition with the differential principle.

One of the interesting paradoxes of this question is the fact that stable regulation, or freedom from vibration, becomes easier to obtain as the inertia effects get worse. In other words, a surge tank of given area will approach the dead-beat wave as the length of the pipe line and the velocity increase. The height of such a tank would have to increase with the length of the pipe line and with the velocity, but unless the area were correspondingly decreased, the oscillation would become steadier as the inertia effects became worse. The reason for this is that the work which may be accomplished within a surge tank is proportional to the square of the height through which the water may be permitted to vary, and this height, for equivalent stability, may vary directly as the friction gradient, which itself varies directly as the length of the pipe line and about as the square of the velocity. The work required to be done, however, varies only directly as the friction gradient, which is to say, also, as the length of the pipe line and as the square of the velocity. Hence, doubling either of these quantities would double the permissible height for water variation within the tank, and would thereby multiply its capacity for work by four. This brings out forcibly the important fact that

very long pipe lines need not be avoided through any misconceived fear of dangerous oscillations of the water column. The unavoidably large change in gradient following a load change is, to be sure, a somewhat undesirable attribute of a very long pipe line, because it causes the governor to travel a little farther than it would if the head did not vary, but no more serious effect than this comparatively mild one need be apprehended if the regulator is designed intelligently.

Some people have argued against the differential action because it causes a more rapid initial variation in head than the simple surge tank, but the fact is that the small capacity furnished by the internal stand-pipe of the differential regulator is quite sufficient to afford just as good practical results as a simple tank of like dimensions, and the manifest advantages in all other respects ought to make it the preferred type in all cases. The writer has never yet found a condition where he could see any advantage in the use of a simple surge tank, and long ago he abandoned the idea entirely, as always extravagant, often impracticable, and sometimes dangerous.

Another important function of the internal stand-pipe is to afford a relief overflow into the tank in case of a sudden shut down, which causes the water to rise immediately to the top of the stand-pipe, and, while spilling into the tank, the conduit receives the benefit of this suddenly applied back pressure which cuts down to a minimum the total quantity of water to be caught. For ordinary load changes, the water merely rises or falls a moderate distance within the stand-pipe, without, of course, overflowing, and the effect of the restricted opening is to force a more rapid variation of level in the stand-pipe than takes place in the surrounding tank, but the restriction is ordinarily not sufficient to cause a greater total variation of level in the stand-pipe than subsequently takes place in the tank due to the transfer of water through the restricted passage. The object of this restriction is to cause a greater acceleration than would result from the comparatively sluggish movement of the water surface in a simple tank, thereby minimizing the quantity of water to be handled by the tank and greatly increasing its ability to check or choke harmful and unnecessary surges. The size of a simple tank to produce the same net results as to regulation of the water-wheel is sometimes enormously larger than that of a differential regulator, the exact

ratio depending on the conditions to be met, and this is especially evident when the test of synchronous load changes is applied.

The ideal differential action to which the following equations apply needs a careful description in order to forestall any misconception as to the limitations of the mathematics, and to set forth clearly the general fundamental conditions which make possible at all a rigid mathematical treatment of the surge phenomena. Certain assumptions are necessary, which are not strictly adhered to in the final design but may be modified with little additional labor when the action of the regulator is fully understood.

First.—The stand-pipe area is considered to be so small that a variation in water level within it takes place in a negligible time compared to the remaining time in which complete acceleration of the water column is effected.

Second.—The restricted opening is assumed to vary in size according to certain laws, with the result that, when a sudden change in level takes place within the stand-pipe (due to a sudden load change), the water level remains absolutely stationary at the new position until complete acceleration is effected in the conduit.

Third.—The size of the tank is required to be such that the ultimate change in water level in one direction shall be exactly equal to the sudden change in the stand-pipe for the given load change under consideration.

Fourth.—The inertia of the water in the regulator itself is neglected.

Fifth.—Velocity changes only, rather than load changes, are considered, and therefore the head, H, is not involved in the theory; it enters only in an extraneous computation to convert velocity change into load change.

Sixth.—The friction in the stand-pipe, velocity head, and the weir head when spilling over the top, are neglected.

Seventh.—The kinetic energy in the flowing water is computed from the average velocity, $\frac{Q}{A}$, and is not refined to take into account the slight error introduced by reason of the non-uniform velocity existing in the cross-section of a cylinder of flowing water.

The first assumption is never actually realized, except where compressed-air tanks are used.

The second assumption is seldom, if ever, actually necessary, because, as will appear later, a constant size of choke hole is quite adequate for practical results, especially when assisted by some capacity in the stand-pipe which has to be neglected for the sake of the integral calculus.

As has been stated before, the following mathematical treatment in its entirety deals only with the phenomena which take place during the first quarter cycle of the water wave; that is to say, during the period of the first programme of acceleration, positive or negative, while the water level in the tank is changing in one direction and that in which it has been started by a load change. The subsequent plotting of the various waves of pressure and velocity must be carried on by recourse to the principles of arithmetic integration which any one who has the patience and is sufficiently interested may do with no more than an ordinary understanding of the fundamental laws of Newton and a good training in the rapid use of a slide-rule. It is not the present purpose to describe these methods in detail. because satisfactory results may be obtained in many ways, and the individual may often select the method best suited to his mind. Suffice it to say that although the mathematical treatment herein set forth is as complete as the writer has been able to make it, yet when the design of a regulator is approximated as closely as possible from the available equations, it is usually much more satisfactory and profitable to one lacking experience, to integrate the phenomena second by second in the common-sense way than to rely solely on judgment for the effect of the departures which have to be made from the hard and fast mathematical assumptions and results.

The ideal differential action to which the equations apply is necessarily confined to one particular load change, and if mathematically ideal conditions were required for a greater load change, the size of the choke hole would have to be increased. It follows, therefore, that the design should be made ample in the first place for the greatest load change for which good regulation is desired, and this should be controlled by the size of the greatest ordinary daily sudden variation when the operation of the plant is regular and normal. If there occurs a greater load change than that for which the regulator has been designed, the result will be a greater total variation of pressure level within the stand-pipe than is necessary or than subsequently takes

place in the tank, and this condition is not desirable as an ordinary operating feature. If, however, abnormal conditions obtain for a time, such as synchronous load changes or excessively large ones, this disproportionate variation in water level within the stand-pipe is an excellent thing, because it serves to counteract and deaden any tendency toward an increasing or unstable vibration of the water level. In other words, the differential action should be harmless under normal conditions, if properly designed, and should be particularly beneficial for abnormal conditions. The advantage of this principle, as distinguished from the simple tank, is not nearly so apparent during the first quarter cycle as it is later in the history of the wave, but even at first it has one decided advantage in that the total variation of head on the wheel is considerably less than in a simple tank of equal size; this permits a smaller water-wheel to adhere to the load following a load change without reaching full gate, or conversely, permits a wheel of the same size to deliver the new increased load at smaller gateage, which means at better efficiency if near full gate, and which, therefore, means less acceleration required in the long conduit, and better conditions all around.

Proper characteristics of the water-wheel itself are of great importance in assisting the surge tank, and particular attention should be paid to the water-wheel specifications, if the surge tank is not to be unreasonably taxed; and, if the wheels are already installed when the task of providing a suitable regulator is undertaken, a knowledge of the water-wheel characteristics is essential for intelligent design. While the efficiency is increasing with the load, regulation is comparatively easy, but, beyond the crest of the efficiency curve, the regulator has additional work to do, and in some instances of excessive friction in a long conduit, further opening of the gates may actually produce less power, which is a case impossible for any regulator to improve.

Nomenclature.—The nomenclature herein adopted is as follows:

t = time in general;

t_a = time of acceleration;

t_r = time of retardation;

T = time to complete one quarter cycle of the water wave;

L = length of the conduit;

A = area of the conduit;

F = net area of tank, that is, in excess of the stand-pipe area;

a = area of restricted opening or port with 100% coefficient of discharge;

a_0 = area of restricted opening when $t = 0$;

a_1 = area of restricted opening when $t = T$;

v = velocity in the conduit;

v_1 = initial conduit velocity before acceleration begins ($t_a = 0$), or = conduit velocity when $t_r = T$, or = constant-draft velocity between surge tank and water-wheel, in terms of the conduit velocity during retardation;

v_2 = conduit velocity when $t_a = T$, or = initial conduit velocity before retardation begins ($t_r = 0$), or = constant-draft velocity between surge tank and water-wheel in terms of the conduit velocity during acceleration;

c = a coefficient such that cv^2 = total losses of head in the conduit (these losses may or may not include the velocity head, depending on the location of the surge tank);

y = departure of the water level in the tank from its initial quiescent position previous to a load change;

$y_1 = y$ for $t = T$ = also, by hypothesis, the amount of the initial sudden change of level in the stand-pipe;

p = percentage of velocity change $= \dfrac{v_2 - v_1}{v_2}$ in acceleration;

$r = \dfrac{v_1}{v_2} = 1 - p$;

k = stability factor of the water wave $= \dfrac{y_1}{c\,(v_2{}^2 - v_1{}^2)}$;

log. = natural logarithm, where not specified;

$$Z = \sqrt{\frac{y_1}{c} + v_1{}^2}, \text{ or } y_{1_a} = c\,(Z^2 - v_1{}^2);$$

$$Z_1 = \sqrt{v_2{}^2 - \frac{y_1}{c}}, \text{ or } y_{1_r} = c\,(v_2{}^2 - Z_1{}^2);$$

$$Z_0 = \sqrt{\frac{y_1}{c} - v_2{}^2}, \text{ or } y_{1_r} = c\,(v_2{}^2 + Z_0{}^2);$$

$$X = \frac{Z}{v_2} = \sqrt{k\,(1 - r^2) + r^2}.$$

ACCELERATION.

By hypothesis we have, for positive acceleration,

$$dt = \frac{\frac{L}{g} d\,v}{y_1 - c\,(v^2 - v_1^{\,2})}$$

from which, by integration between the limits v_1 and v, we have the general expression for the time which has elapsed in acquiring any velocity

$$t_a = \frac{L}{2\,g\,c\,Z} \log. \frac{(Z - v_1)\,(Z + v)}{(Z + v_1)\,(Z - v)} \ldots\ldots\ldots\ldots (1)$$

Multiplying both sides of the differential equation by $A\ v$, we have

$$A \int_0^t v\,dt = \frac{A\ L}{g} \int_{v_1}^{v} \frac{v\ dv}{y_1 - c\,(v^2 - v_1^{\,2})}$$

which equals the total quantity of water discharged through the conduit in the time, t. Now, by hypothesis, the total quantity drawn through the water-wheels in the time, t, is $A\ v_2\ t$, and the difference must be taken from the tank; whence,

$$A\,v_2\,t_a - \frac{A\ L}{g} \int_{v_1}^{v} \frac{v\ dv}{y_1 - c\,(v^2 - v_1^{\,2})} = y\,F.$$

Integrating and simplifying, and substituting the value of t_a from Equation (1), we have

$$y = \frac{A\ L}{2\,g\,c\,F} \left\{ \frac{v_2}{Z} \log. \frac{(Z - v_1)\,(Z + v)}{(Z + v_1)\,(Z - v)} - \log. \frac{Z^2 - v_1^{\,2}}{Z^2 - v^2} \right\} \ldots (2)$$

This is the general expression giving the momentary relation between the drop in water level in the tank and the corresponding conduit velocity which has been attained.

When v becomes equal to v_2, y becomes equal to y_1, if F is of the correct size, hence we may place $y = y_1$, and solve for F. Remembering that

$$T_a = \frac{L}{2\,g\,c\,Z} \log. \frac{(Z - v_1)\,(Z + v_2)}{(Z + v_1)\,(Z - v_2)} \ldots\ldots\ldots\ldots (3)$$

we have, for correct size of tank,

$$F = \frac{A\ L}{2\,g\,c\,y_{1_a}} \left\{ \frac{v_2}{Z} \log. \frac{(Z - v_1)\,(Z + v_2)}{(Z + v_1)\,(Z - v_2)} - \log. \frac{Z^2 - v_1^{\,2}}{Z^2 - v_2^{\,2}} \right\} \ldots (4)$$

or, in other notation,

$$F = \frac{A\ L}{2\,g\,c^2\,v_2^{\,2}\,k\,(1 - r^2)} \left\{ \frac{1}{X} \log. \frac{(X - r)\,(X + 1)}{(X + r)\,(X - 1)} - \log. \frac{k}{k - 1} \right\} (5)$$

from which F may be solved directly for any velocity change and any degree of stability, k, desired. Equation (5), at first sight, may seem to be a useless algebraic contortion of Equation (4), but it may be found instructive to have due regard to k rather than the more direct value y_1 which in itself gives no adequate idea of how much the motion of the water level exceeds the gradient change.

The head, h_p, acting on the restricted passage or "port" at any time $= y_1 - y$, and may be expressed in terms of the corresponding velocity as follows:

$$y_1 - \frac{A\,L}{2\,g\,c\,F}\left\{\frac{v_2}{Z}\log.\frac{(Z-v_1)(Z+v)}{(Z+v_1)(Z-v)} - \log.\frac{Z^2-v_1^2}{Z^2-v^2}\right\}$$

or, substituting the value of y_1 in terms of Z from Equation (4), we have

$$h_p = \frac{A\,L}{2\,g\,c\,F}\left\{\frac{v_2}{Z}\log.\frac{(Z-v)(Z+v_2)}{(Z+v)(Z-v_2)} - \log.\frac{Z^2-v^2}{Z^2-v_2^2}\right\} \quad \text{..(6)}$$

The quantity of water necessary to be forced through the port by the head, h_p, is $A\,(v_2 - v)$; therefore,

$$a\,\sqrt{2\,g\,h_p} = A\,(v_2 - v)$$

and

$$a = \frac{\sqrt{A}\,(v_2 - v)}{\sqrt{\frac{L}{c\,F}\left\{\frac{v_2}{Z}\log.\frac{(Z-v)(Z+v_2)}{(Z+v)(Z-v_2)} - \log.\frac{Z^2-v^2}{Z^2-v_2^2}\right\}}} \quad \text{..(7)}$$

which is the general expression for the size of the port for any assigned conduit velocity, during a surge.

When $v = v_1$ at the beginning of the cycle,

$$a_0 = \frac{A\,(v_2 - v_1)}{\sqrt{2\,g\,y_1}} \quad \text{.......................(8)}$$

When $v = v_2$, the expression becomes indeterminate, but when evaluated by differentiating the numerator and denominator, we find

$$a_1 = \left\{\frac{A\,F\,y_1}{L}\left(1 - \frac{1}{k}\right)\right\}^{\frac{1}{2}} \quad \text{...............(9)}$$

which is always less than a_0.

It is interesting to note the extreme simplicity of the foregoing equations when there is considered to be no friction in the conduit. A single port of fixed area is then found to be exactly adequate to hold the accelerating water level at one point while the conduit velocity

is attaining a value which satisfies the new load. In other words, when the tank has the correct area, F, the pressure head on the fixed port which exists at any instant is just that required to force through it the difference between the quantity of water demanded by the water-wheel and that being furnished, at that particular instant, by the conduit.

By hypothesis, we have

$$dt = \frac{L}{g\,y_1}\,dv \text{ and } dy = \frac{A}{F}\,(v_2 - v)\,dt.$$

Eliminating dt, we have

$$dy = \frac{A\,L}{F\,g\,y_1}\,(v_2 - v)\,dv,$$

from which, by integration between the limits v_1 and v, we obtain

$$y = \frac{A\,L}{2\,g\,F\,y_1}\left\{(v_2 - v_1)^2 - (v_2 - v)^2\right\} \quad\ldots\ldots\ldots\ldots(10)$$

also, similarly,

$$t = \frac{L}{g\,y_1}\,(v - v_1) \quad\ldots\ldots\ldots\ldots\ldots\ldots(11)$$

$$y_1 = \frac{A\,L}{2\,g\,F\,y_1}\,(v_2 - v_1)^2 \text{ and } T = \frac{L}{g\,y_1}\,(v_2 - v_1) = \sqrt{\frac{2\,L\,F}{g\,A}}$$

$$F = \frac{A\,L}{2\,g\,y_1^2}\,(v_2 - v_1)^2.$$

These equations hold good for both acceleration and retardation. Expressing the area of port in terms of v and y we have, as before,

$$a^2 = \frac{A^2\,(v_2 - v)^2}{2\,g\,(y_1 - y)},$$

or, substituting the values of y_1, and y in terms of v,

$$a^2 = \frac{A\,F\,y_1\,(v_2 - v)^2}{L\,\left\{(v_2 - v_1)^2 - (v_2 - v_1)^2 + (v_2 - v)^2\right\}} = \frac{A\,F\,y_1}{L} = \text{constant},$$

or,

$$a = \sqrt{\frac{A\,F\,y_1}{L}} = \text{constant} \quad\ldots\ldots\ldots\ldots\ldots\ldots(12)$$

Equation (12) is the limit of Equation (7) as c approaches zero.

The value of a_0 for $c = 0$ may also be shown to be the same, by substituting for $(v_2 - v_1)$ in Equation (8) its value as just determined in terms of y_1, and we again have

$$a_0 = \sqrt{\frac{A\,F\,y_1}{L}}.$$

Also, since $\frac{1}{k}$ becomes zero, Equation (9) becomes

$$a_1 = \sqrt{\frac{A\,F\,y_1}{L}}.$$

The port area remains constant throughout the quarter cycle.

Equations (10), (11), and (12) are useful when c is so small as to make the previous formulas too sensitive to solve with ordinary logarithmic tables. Equation (12), as compared with Equation (7), shows that the less the friction the less the variation in a in order to hold a perfectly stationary accelerating water level in the stand-pipe. This variation in the size of effective port may be approximated as closely as desired by a series of holes up and down the stand-pipe, which would shut off automatically as the water dropped below them in the tank, but this refinement is ordinarily not warranted, and when there is some appreciable capacity in the stand-pipe, as for example, if it has an area $= A$, a single port of fixed size is quite sufficient for all practical purposes except for very unusual conditions. No serious error would be made in fixing the size of the port by Equation (8), but a trial computation by arithmetic integration for any particular case would show that a somewhat smaller hole than a_0 gives slightly better results. The reason it is not usually necessary to refine too carefully the dimensions of the port, is that, if one uses Equation (8), the port will always be somewhat too large, which is the same thing as saying that a somewhat larger velocity change than the one assumed would most nearly approach the mathematical ideal. No particular harm results from making the hole a little too large, although it should be as nearly right as possible, for as it gets larger the dangers of the simple tank are being approached.

When k approaches unity, Case (5), or the dead-beat wave, is approximated, and for $k = 1$, the water level does not pass beyond the final quiescent position for the increased load.

When $k = 1$, $Z = v_2$, and Equation (1) becomes for dead-beat conditions,

$$t_a = \frac{L}{2\,g\,c\,v_2} \log. \frac{(v_2 - v_1)\,(v_2 + v)}{(v_2 + v_1)\,(v_2 - v)} \quad \ldots\ldots\ldots\ldots (13)$$

and when $v = v_2$, $T_a = \infty$.

Equation (2) is much simplified by substitution of $Z = v_2$ and combining the logarithms, and becomes, for dead-beat conditions,

$$y = \frac{A\,L}{g\,c\,F} \log. \frac{v_2 + v}{v_2 + v_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

and when $v = v_2$ and y becomes y_1, we have

$$y_1 = \frac{A\ L}{g\ c\ F} \log. \frac{2\ v_2}{v_2 + v_1} = c\ (v_2^2 - v_1^2)$$

from which

$$F_d = \frac{A\ L}{g\ c^2\ (v_2^2 - v_1^2)} \log. \frac{2\ v_2}{v_2 + v_1} \ldots\ldots\ldots\ldots\ldots (15)$$

Also, when $k = 1$, $Z = v_2$ and Equation (7) becomes, for dead-beat conditions,

$$a = \frac{\sqrt{A}\ (v_2 - v)}{\sqrt{\frac{2\ L}{F\ c} \log. \frac{2\ v_2}{v_2 + v}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Equation (8) remains unchanged, except for the possible substitution of $y_1 = c\ (v_2^2 - v_1^2)$; and Equation (9) becomes $a_1 = 0$.

It has been pointed out previously that the extent of the variation in a depends on the amount of the friction, and the greatest variation from a_0 to a_1 is found when $a_1 = 0$, for the condition of perfect damping effect. If this ideal result were required to be practically obtained with a tank as small as given by Equation (15), the stand-pipe would need to contain an infinite number of holes, which would merge into a slot of peculiar shape, but this is an unnecessary amount of refinement because absolute damping is very little better than a near approach to it, and, besides, a moderately larger tank could be made to accomplish it with far less refinement of the port area. One fixed hole should usually be sufficient in all cases.

The limiting value of F for small load changes is interesting, particularly because as v_2 approaches v_1 in value, the solution of previous equations becomes troublesome, and, for the complete plotting of a curve, for example, the limit must be known. It might appear at first thought that when the velocity change is zero the tank area would also be zero, but for a given required stability, k, the value of F approaches a perfectly definite limit as r approaches unity. The value of a, however, does approach zero. The limit of $\frac{(X - r)\ (X + 1)}{(X + r)\ (X - 1)}$, as r approaches unity, is $\frac{k}{k - 1}$; therefore Equation (5) is a true indeterminate with respect to F, and by multiplying numerator and denominator by X and performing the necessary successive differentiations we find

$$F_0 = \frac{A\ L}{4\ g\ c^2\ v^2\ k} \left\{ 1 - (k - 1) \log. \frac{k}{k - 1} \right\} \ldots\ldots\ldots (17)$$

The value of F_0 for dead-beat conditions is also useful to assist in solving equations which approach the indeterminate form as k approaches unity.

The limit of $(k-1)$ log. $\frac{k}{k-1}$ as k approaches unity is zero, and therefore

$$F_{a_0} = \frac{A\,L}{4\,g\,c^2\,v^2} \quad \text{.......................(18)}$$

This completes the study for positive acceleration, so far as the writer has carried it up to the present time, and the variations which occur for retardation will now be considered.

Retardation.

By hypothesis, we have for retardation

$$dt = \frac{\frac{L}{g}\,dv}{y_1 - c\,(v_2^2 - v^2)}.$$

In the corresponding equation for acceleration the constant term in the denominator was always positive and hence the integration took but one form. In this case, however, y_1 does not always reach forebay level for part load rejected, and therefore may have a value either greater or less than cv_2^2, thus causing the constant term in the denominator to vary through zero from negative to positive as y_1 is increased.

(1) When Water Does Not Reach Forebay Level on Rejection of Load.—When $y_1 < c\,v_2^2$, we have, as for Equation (1),

$$t_r = \frac{L}{2\,g\,c\,Z_1} \log. \frac{(v_2 - Z_1)\,(v + Z_1)}{(v_2 + Z_1)\,(v - Z_1)} \quad \text{.............(19)}$$

and, as a companion of Equation (2), we have, similarly,

$$y_r = \frac{A\,L}{2\,g\,c\,F} \left\{ \log. \frac{v_2^2 - Z_1^2}{v^2 - Z_1^2} - \frac{v_1}{Z_1} \log. \frac{(v_2 - Z_1)\,(v + Z_1)}{(v_2 + Z_1)\,(v - Z_1)} \right\} \text{...(20)}$$

and for the value of F we have

$$F = \frac{A\,L}{2\,g\,c y_{1r}} \left\{ \log. \frac{v_2^2 - Z_1^2}{v_1^2 - Z_1^2} - \frac{v_1}{Z_1} \log. \frac{(v_2 - Z_1)\,(v_1 + Z_1)}{(v_2 + Z_1)\,(v_1 - Z_1)} \right\} \text{..(21)}$$

For the case when $y_1 = cv_2^2$, which means that a certain critical value of F just forces the water to forebay level for an assigned value

of $v_2 - v_1$, we find that Equation (19) becomes indeterminate as Z_1 approaches zero. On evaluation by differentiation we have

$$t_r \text{ (for } Z_1 = 0) = \frac{L}{g\ c} \times \frac{v_2 - v}{v\ v_2} \quad \text{.............. (22)}$$

and Equation (20) becomes

$$y \text{ (for } Z_1 = 0) = \frac{A\ L}{g\ c\ F} \left\{ \log. \frac{v_2}{v} - v_1 \frac{v_2 - v}{v\ v_2} \right\} \quad \text{....... (23)}$$

$$\text{and } F \text{ (for } Z_1 = 0) = \frac{A\ L}{g\ c^2 v_2{}^2} \left\{ \log. \frac{v_2}{v_1} - \frac{v_2 - v_1}{v_2} \right\} \quad \text{....... (24)}$$

When c approaches zero, the equations for retardation are exactly the same as already derived for acceleration, namely, Equations (10), (11), and (12).

The foregoing equations for retardation are chiefly interesting from a mathematical standpoint because in a practical case very little attention need be paid to what happens when part load is rejected, so long as a complete shut-down is properly taken care of. Therefore, the size of the port is controlled by the acceleration studies, and is limited in size only by the shut-down conditions, as will appear later. The equations for dead-beat conditions in retardation may as well be written, for the sake of comparison with Equations (14) and (15). By processes strictly analogous to the previous cases, we have for dead-beat retardation, which can exist only for rational values of $Z_1 = v_1$

$$y = \frac{A\ L}{g\ c\ F} \log. \frac{v_2 + v_1}{v_1 + v} \quad \text{.................... (25)}$$

and when $v = v_1$ and y becomes y_1, we have as compared to Equation (15)

$$F = \frac{A\ L}{g\ c^2\ (v_2{}^2 - v_1{}^2)} \log. \frac{v_2 + v_1}{2\ v_1} \quad \text{.............. (26)}$$

It will now be seen that F in Equation (26) is always larger than F in Equation (15), because $\frac{v_2 + v_1}{2\ v_1} > \frac{2\ v_2}{v_2 + v_1}$, and, although full load may be thrown on without producing oscillation, in which case $\frac{2\ v_2}{v_2 + v_1}$ becomes $= 2$ for $v_1 = 0$, yet oscillation is bound to occur for full load rejected, except for an infinite value of F, because $\frac{v_2 + v_1}{2\ v_1}$ becomes ∞ for $v_1 = 0$.

In other words, the water must in all cases reach a level higher than the forebay. The value of F for dead-beat conditions for very small load changes, however, would be identical with that given by Equation (18), because the limit of log. $\frac{v_2 + v_1}{2 v_1} \div (v_2^2 - v_1^2)$ as v_2 becomes $= v_1$, is $\frac{1}{4 v_1^2}$.

When Water Rises Above Forebay on Rejection of Load.—The phenomena resulting from irrational values of Z_1 will now be taken up. This is by far the more usual case in practice, where the water rises above forebay level following a rejection of part load. As Z_1 becomes a surd, it is no longer of any use, and it must be supplanted by $Z_0 = \sqrt{\frac{y_1}{c} - v_2^2}$. Reverting to the original differential equation,

$$dt = \frac{\frac{L}{g} d v}{y_1 - c\,(v_2^2 - v^2)}$$

we find that when $y_1 > c\, v_2^2$, the integration takes the form of the $\tan.^{-1}$, instead of the log., and we have

$$t_r = \frac{L}{g\, c\, Z_0} \left\{ \tan.^{-1} \frac{v_2}{Z_0} - \tan.^{-1} \frac{v}{Z_0} \right\} \dots\dots\dots\dots (27)$$

and

$$y = \frac{A\, L}{2\, g\, c\, F} \left\{ \log. \frac{v_2^2 + Z_0^2}{v^2 + Z_0^2} - \frac{2\, v_1}{Z_0} \left(\tan.^{-1} \frac{v_2}{Z_0} - \tan.^{-1} \frac{v}{Z_0} \right) \right\} (28)$$

and

$$F = \frac{A\, L}{2\, g\, c\, y_1} \left\{ \log. \frac{v_2^2 + Z_0^2}{v_1^2 + Z_0^2} - \frac{2\, v_1}{Z_0} \left(\tan.^{-1} \frac{v_2}{Z_0} - \tan.^{-1} \frac{v_1}{Z_0} \right) \right\} (29)$$

and evaluating Equation (29) for $Z_0 = 0$ we again have Equation (24), which shows that Equations (29) and (21) draw together, as they should, and become identical for the condition that forebay level shall exist in the tank when $t_r = T_r$.

The most useful equations for rejected load are those pertaining to a shut-down when $v_1 = 0$, and Equation (28) becomes

$$y = \frac{A\, L}{2\, g\, c\, F} \log. \frac{v_2^2 + Z_0^2}{v^2 + Z_0^2} \dots\dots\dots\dots\dots\dots (30)$$

and Equation (29) becomes

$$F = \frac{A\, L}{2\, g\, c\, y_1} \log. \frac{v_2^2 + Z_0^2}{Z_0^2} \dots\dots\dots\dots\dots\dots (31)$$

and, as $k_r = \dfrac{y_1}{c\, v_2^{\,2}}$, we may rewrite Equation (31) in terms of k_r as follows:

$$F = \frac{A\, L}{2\, g\, c^2\, v_2^{\,2}\, k_r} \log. \frac{k_r}{k_r - 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

from which k_r may be determined by trial, provided F has been previously fixed by the consideration of demanded load. It sometimes happens, however, that F is fixed by the maximum hydrostatic pressure desirable which determines k_r, in which case the value of a for demanded load is subject to adjustment to a given value of F.

The limiting size of the port hole, a_1, for a shut-down which obtains when $t_r = T_r$, just as the water comes to rest in its upward movement in the tank at the level of the top of the stand-pipe, is determined by processes analogous to those used in arriving at Equation (9) and becomes

$$a_1 = \left\{ \frac{A\, F\, y_1}{L} \left(1 - \frac{1}{k_r}\right) \right\}^{\frac{1}{2}}$$

or,

$$a_1 = v_2 \left\{ \frac{A\, F\, c}{L} \left(k_r - 1\right) \right\}^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots (33)$$

If k_r has been determined by trial from Equation (32), its value may be substituted in Equation (33) and the limiting value of a derived. The size of the port hole, as determined for demanded load, must not exceed this limiting value, for otherwise the water, having reached the top of the stand-pipe, following a shut-down, will not remain there for the time, T, but will shrink back down the stand-pipe during the quarter cycle, afterward rising again, and thus will produce a condition contrary to the hypothesis of a constant value of y_1, and may introduce an error of some magnitude. It happens, fortunately, that the requirement of design for excessively large demanded loads is rare, and almost always, when the value of a is decided on for demanded loads, it is incidentally smaller than the limiting value given by Equation (33). This will always be the case when, in the acceleration studies, $2\, r$ is not less than

$$c\, v_2 \sqrt{\frac{2\, g\, F}{A\, L}} + \frac{1}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

which has been determined by graphical methods, but should be verified by trial for any particular case. It will be observed that the less the friction, other things remaining unchanged, the greater the load change

for which the capacity of the port will provide without exceeding a suitable size for a shut-down; and when $c = 0$, the same value of a would be suitable, either for throwing on or off the full load. The factor, $\frac{1}{10}$, in Equation (34) is merely empirical.

In selecting the value of v_2, to determine the maximum height of the tank, it must be borne in mind that a larger value is likely to be required than the value of v_2 found for a maximum load demanded, because the conduit velocity somewhat exceeds the value existing when $t_a = T$, except for dead-beat conditions, and a shut-down could occur just at the critical point of maximum conduit velocity during the second quarter cycle following an increment of load. The extent of this excess may be left to the judgment, or it may easily be roughly determined by a few steps of arithmetic integration. It is really so small a matter as compared to the uncertainty attending the selection of a low enough value of c, that the judgment is usually quite sufficient, and, furthermore, it does not always follow that higher velocities call for higher surge tanks to catch all the water, for, as will now be demonstrated, there exists for all conditions, except the hypothetical one where $c = 0$, a critical velocity which necessitates a higher surge tank than any greater or smaller velocity. This critical velocity, of course, may be greater than that due to full load, but it frequently occurs at part load for large values of c or L, and in general for values of k_a near unity.

The height above forebay level to which the stand-pipe and tank must be carried to provide sufficient capacity to catch all the energized water following a shut-down, let us call d, and its value is evidently

$$d = c\ Z_0{}^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

and it is required to find the value of v_2 which makes d a maximum; and as c is constant for any particular case, d will be a maximum when Z_0 is a maximum.

Putting $y_1 = c\ (Z_0{}^2 + v_2{}^2)$ in Equation (31), we have

$$\frac{2\,g\,c^2\,F}{A\,L} = \frac{\log.\ (v_2{}^2 + Z_0{}^2) - 2\log.\ Z_0}{Z_0{}^2 + v_2{}^2}$$

from which we may find the value of v_2 when $\frac{d\,Z_0}{d\,v_2} = 0$, which we shall call v_c, and we have

$$v_c = \frac{1}{c}\left\{\frac{A\,L\,(e-1)}{2\,g\,F\,e}\right\}^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (36)$$

in which e is the base of the natural logarithms and v_c becomes a little less than

$$\frac{1}{10\,c}\sqrt{\frac{A\,L}{F}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(37)$$

If the value of v_2 which is chosen as a maximum to determine the greatest height of tank and stand-pipe is greater than v_c, we must conclude that a shut-down from some part load will require a higher elevation to avoid spilling. In many cases v_c may not be so much above v_2 as materially to increase the height of tank if the former value is assumed, even though we do not expect quite so high a value, and, where economically feasible, it is best to provide for the worst case chiefly because the absolute maximum velocity which may at some future time exist coincident with a shut-down is a difficult thing to fix with precision. It is usually economically feasible to do this when the value of c is high, as is commonly the case for very long pipe lines. When not commercially desirable to do this, the value of $\frac{v_2}{c}$ must be made large enough to cover all reasonable contingencies. The value of d for $v_2 = v_c$ may be expressed by solving for $d = c\,Z_0^2$ in Equation (31) and we have

$$d_{max.} = \frac{A\,L}{2\,g\,c\,e\,F}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(38)$$

which is the greatest distance above forebay level which a tank of area F need ever be carried for any velocity whatsoever. Perhaps the most striking feature of this equation is its independence of the value of L. This is evident because c is proportional to L. The limiting value of a_1 for v_c is obtained from Equations (33), (36), and (38), namely,

$$a_1\,(\text{for } v_c) = \frac{A}{\sqrt{2\,g\,c\,e}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(39)$$

Equation (33) may also be written

$$a_1 = \sqrt{\frac{A\,F\,d}{L}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(40)$$

This concludes the study in its present state of development, and it is to be hoped the critics will add something to it. Before closing it is desired to call attention to the kinematic relation which ties together all the foregoing equations.

For a given value of p and k in Equation (5), the number represented by $\frac{2\,g\,c^2\,v_2^2\,F}{A\,L}$ is constant $= N^2$ (say) .... (41)

That is, if a certain fixed percentage velocity change, p, is desired together with an assigned fixed degree of stability, k, then any mutual interchange in the values of Equation (41) may be made without affecting the result. This is very important because each assignment of p and k covers an infinite variety of combinations without further computation except to manipulate Equation (41) in any conceivable manner so long as the ratio there expressed remains constant. As c varies approximately as $\frac{L^*}{A^{\frac{3}{5}}}$, we may more properly write, in this connection, $c = \frac{f\,L}{A^{\frac{3}{5}}}$ where f is the coefficient of friction, and Equation (41) becomes, omitting the constants, 2 g and f,

$$\frac{v_2^2\,L\,F}{A^{2.2}} = \text{constant}.$$

Also, similarly, $\frac{a\,\sqrt{L}}{A^{1.3}} = \text{constant}$, as $\frac{a\,\sqrt{c}}{A}$ is constant in Equations (8), (9), etc., for fixed values of p and k.

L and F may be varied inversely with respect to each other, v and A remaining constant, or any other similar interchange may be made without affecting the kinematic relation of the design to the assigned requirements.

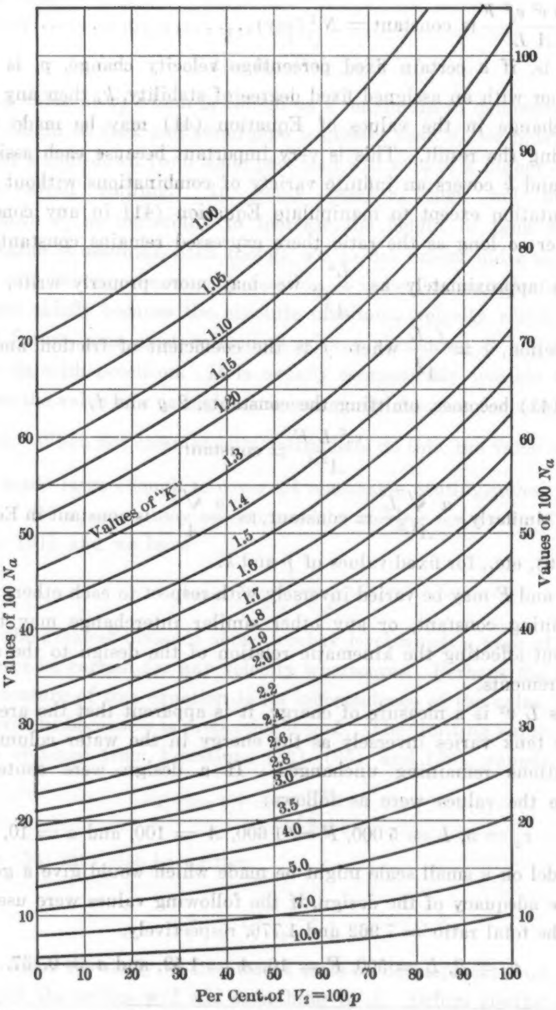
As $L\,v^2$ is a measure of energy, it is apparent that the area of the surge tank varies inversely as the energy in the water column, other conditions remaining unchanged. If a design were contemplated where the values were as follows:

$$v_2 = 5,\ L = 5\,000,\ F = 1\,600,\ A = 100, \text{ and } a = 10,$$

a model on a small scale might be made which would give a good idea of the adequacy of the design, if the following values were used, keeping the total ratio = 7 962 and 1.776, respectively,

$$v_2 = 2,\ L = 500,\ F = 10,\ A = 1.52, \text{ and } a = 0.137.$$

*This expression for the variation of c is not intended to represent an accepted law; it is merely illustrative. According to Williams and Hazen, for example, c would vary as $\frac{L}{A^{0.584} \times v^{0.15}}$, and some such suitable refinement would be desirable for actual use.



Values of 100 N_a
Values of 100 N_a
Values of "K"
1.00
1.05
1.10
1.15
1.20
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
2.2
2.4
2.6
2.8
3.0
3.5
4.0
5.0
7.0
10.0
0
10
20
30
40
50
60
70
80
90
100
Per Cent of $V_2 = 100p$

FIG. 1.

This interesting relation of scale suggests a convenient method of treating the entire subject graphically and thus avoiding all trial and error computations, involving cumbersome logarithms and anti-tangent functions, and enabling determinations to be easily made where the equations become sensitive as they approach indeterminate forms. The writer uses graphical methods almost entirely, where all the possible

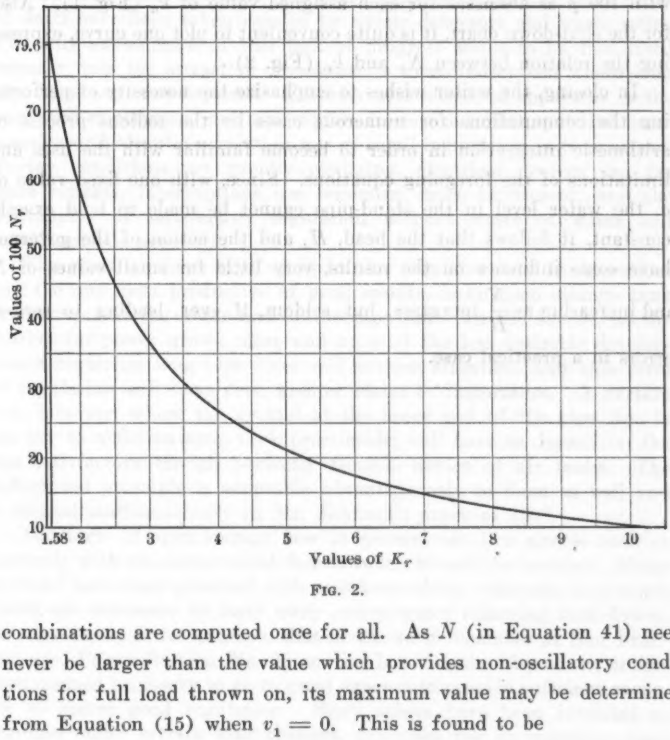


FIG. 2.

combinations are computed once for all. As N (in Equation 41) need never be larger than the value which provides non-oscillatory conditions for full load thrown on, its maximum value may be determined from Equation (15) when $v_1 = 0$. This is found to be

$$N_a \text{ max.} = \sqrt{2 \log. 2} = 1.178 \ldots\ldots\ldots\ldots\ldots (42)$$

Thus it is seen that the value of N in acceleration does not vary through a wide range, and all possible values may easily be contained on one sheet of cross-section paper, for they all lie between 0 and 1.178.

Similarly, for a shut-down, the value of N_r max. may be obtained from Equation (36) in which

$$N_r \text{ max.} = \sqrt{\frac{e-1}{e}} = 0.796 \ldots\ldots\ldots\ldots\ldots\ldots (43)$$

and all the possible values of N_r lie between zero and 0.796, and may also easily be completely contained on a single sheet of cross-section paper. The most convenient method is to plot 100 N_a as ordinates with 100 p as abscissas for each assigned value of k_a (Fig. 1). Also, for the shut-down chart, it is quite convenient to plot one curve, expressing the relation between N_r and k_r (Fig. 2).

In closing, the writer wishes to emphasize the necessity of performing the computations for numerous cases by the tedious process of arithmetic integration in order to become familiar with the uses and limitations of the foregoing equations. Since, with one fixed value of a, the water level in the stand-pipe cannot be made to hold exactly constant, it follows that the head, H, and the action of the governor have some influence on the results, very little for small values of N and increasing as $\frac{N}{H}$ increases, but seldom, if ever, leading to serious errors in a practical case.

DISCUSSION

Roy Taylor,* Esq. (by letter).—In comparison with the great advancement made during recent years in all branches of hydro-electric work, progress in the matter of regulating long pipe lines seems to have remained practically stationary. With the exception of improvements in some details, regulating works to-day are built the same as those of ten years ago. In many cases even the idea of regulating the long water columns has been abandoned, and the fluctuations of draft have been taken care of by nozzle deflectors and waste weirs. The only explanation of this lack of progress seems to be that until recently only the attractive developments were undertaken where long pipe lines were not required. Troubles with electrical apparatus in the past were so common, and interruption in service so frequent that occasional poor hydraulic regulation was of small importance in comparison. To-day conditions are rapidly changing, electrical devices are far more reliable, interruptions are becoming more infrequent, and the desirability of satisfactory regulation without waste of water will soon become a necessity. Mr. Taylor.

The most common type of hydraulic regulator now in operation, and the one most productive of good results, is an open storage tank or reservoir at the lower end of the pipe line. In the future, as the market for power grows, more and more of the less desirable developments requiring long pipe lines will attract attention, and this form of regulation will have even greater claim to importance. A certain few, however, where the ground at the lower end of the pipe line is too low to make an open tank practicable, will have to depend on the less satisfactory though perfectly feasible device of air tanks. The differential principle is adaptable advantageously to them as well and is treated mathematically in Mr. Johnson's paper of 1908.

The form of open storage now in general use is a simple tank or reservoir with an unrestricted flow between it and the conduit. Many of these have been provided with overflows which, although they waste water, are necessary to carry away excess water following shut-downs. In some cases, as for example that of the surge chamber at San Francisquito, Power Station No. 1, on the Los Angeles Aqueduct, it has been deemed necessary to go to great expense to provide sufficient capacity to insure good regulation. Many others have been intended to regulate under certain load changes, yet when put in operation have proved unsatisfactory, due probably to the coincidence of change of load with the periodic surges in the tank. Some find it possible to go on operating under these unstable conditions with poor regulation, and a few have been forced through expensive accidents to remodel their whole regulating works.

* Niagara Falls, N. Y.

Mr. Taylor.

That practically all these disadvantages of the simple tank can be overcome, and the difficulties now arising in operating with some of them can, in a large measure, be relieved by the use of the differential principle, there is not the slightest doubt.

For equal regulating ability under the same load changes, the differential surge tank can be made (1) much smaller in diameter than the simple tank; (2) it does not have to be carried to such height as the latter, often saving an expensive spillway and waste of water; and (3), most important of all, it is proof against synchronous load changes, which, however small, are likely to carry the water in the ordinary simple tank beyond bounds, or at least to cause serious surging. The probability that load will be thrown on and off many times synchronously with the natural period of the water wave in the tank, may be slight, yet, as a possibility, it is always present, especially as the loads may be of almost any size and do not have to come directly in step with the water wave to cause serious surging. An example of the comparative effect of the same synchronous load changes on the operation of a differential and simple tank of equivalent capacities has been computed by the method of arithmetic integration, and is shown graphically in Fig. 3. It may be interesting to know that this diagram is taken from an actual study made for a proposed development. The surge in the simple tank soon increases beyond bounds, but, in the differential tank, it increases slightly until the choking action is enough to prevent further increase in the surging; and, if the same load changes are continued periodically, the surging remains at constant amplitude.

Although Fig. 3 represents very well the usual difference in performance to be found between the two types of tank, it must not be concluded that every simple tank will behave in this manner, for, by increasing the diameter sufficiently, a surging of constant amplitude can be approached, though the cost of this absolute safety is usually prohibitive. The Salmon River development at Altmar, N. Y., in all probability would never have been attempted if the "differential" method of controlling surges in a long pipe line had not been available, for the cost of a simple tank extending perhaps 250 ft. into the air, and sufficiently large to obtain reasonable regulation and safe operation, would not have been commercially feasible. There is no doubt that many simple tanks have been built and are now operating with a large element of risk, not always because the dangers were unknown, but because greater safety would have cost too much.

Fig. 4 represents the difference in performance between a simple and differential tank for one load change. The simple tank has a diameter equal to that of the differential tank and all conditions effecting regulation are identical. The integration has been carried through to show the high surge following the first large downward

Mr. Taylor.

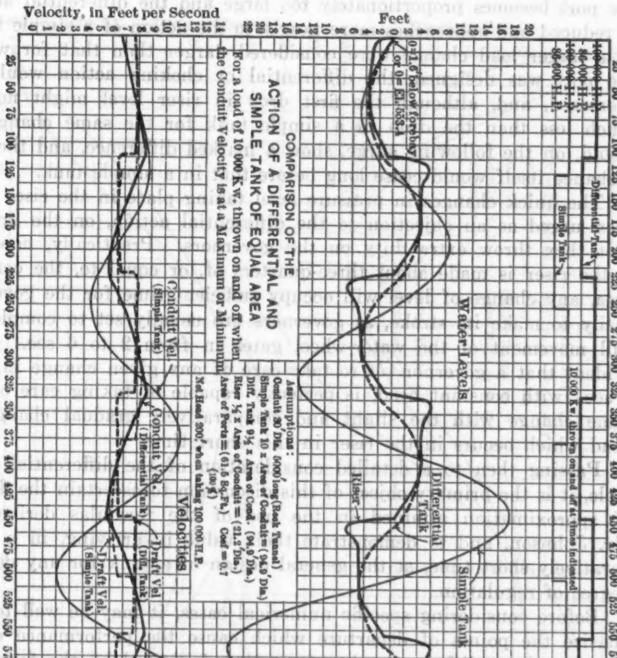


FIG. 3.

Mr. Taylor. dip or to a point beyond three-quarters of a complete cycle. In both cases the surging is decreasing, and the marked difference of 5 ft. appearing between the two in the first quarter cycle is not so pronounced on the following upward wave, for, as the surging dies out, the port becomes proportionately too large and the differential action is reduced and approaches more nearly to the action of a simple tank. If a larger load change were considered, larger than that for which the port was designed, the differential or choking action would be increased, and, although the first drop in riser level might not be much less than the drop in a simple tank for the same change, it would, on the following surge, show a marked difference, and the disturbance itself would cease long before that in a simple tank.

The quick changes in pressure level taking place in the riser have been urged as an objection to the differential action, on the ground that they throw extra duty on the governors. Practically, however, if the riser is made about three-quarters of, or equal to, the conduit area, any change of level will occupy sufficient time for the governor easily to make its stroke, as governors are usually set to complete a full movement of the water-wheel gates in from 2 to 6 sec. It is evident that a governor set to take care of any given change in, say, 4 sec., with constant head, is perfectly capable of taking care of the same change with the slight and comparatively gradual change in head which occurs in the riser in this short time.

Passing then to a detailed consideration of the differential principle, it is the primary object of this discussion to ascertain the degree of approximation attained in the use of the formulas derived by Mr. Johnson, and to demonstrate their value in arriving, in a comparatively short time, at the general design of a tank for any desired degree of regulation.

Before considering specific numerical cases, it may be well to emphasize the points of departure which cause the performance of an actual tank to differ somewhat from that indicated by the formulas. In order to treat the phenomena mathematically, Mr. Johnson has had to assume:

(1) That the riser has zero area, causing the pressure level to vary instantaneously following a load change; and

(2) That the port area varies during the quarter cycle in such a way as to supply to, or receive from, the riser a quantity of water just equal to the difference in flow between that flowing in the pipe line and that drawn by the water-wheel at any instant, and thereby hold the pressure level constant.

These are the essential departures of the theoretical from the actual design, for, in practice, to obtain the best results, the riser is given an area of from three-quarters to the full area of the pipe, although other

Mr. Taylor.

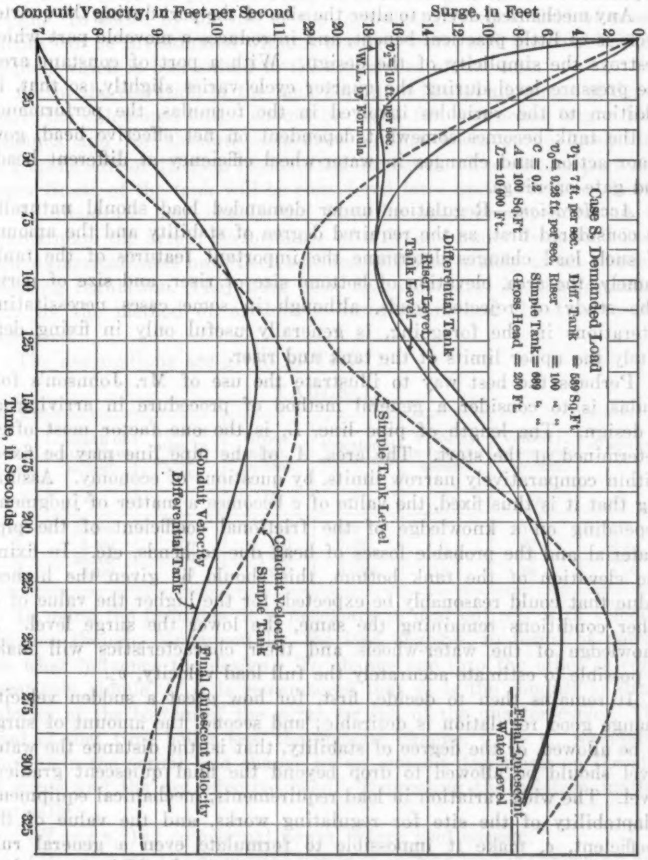


Fig. 4.

Mr. Taylor.

sizes may sometimes be used to advantage, and the port is made as simple as possible, merely a hole or, occasionally, several holes connecting the riser pipe with the tank, at different elevations, and arranged, where possible, with their axes parallel to the riser axis to avoid impact of converging streams, and consequent uncertainty of discharge.

Any mechanical device to alter the size of the port during the quarter cycle is of little practical benefit, and introduces a movable part which destroys the simplicity of the design. With a port of constant area, the pressure level during the quarter cycle varies slightly, so that, in addition to the variables involved in the formulas, the performance of the tank becomes somewhat dependent on net effective head, governor action, and changes in water-wheel efficiency at different heads and gate openings.

Acceleration.—Regulation under demanded load should naturally be considered first, as the required degree of stability and the amount of such load changes determine the important features of the tank, namely, the area, elevation of bottom, size of riser, and size of ports. The study of rejected loads, although in some cases necessitating alterations in the foregoing, is generally useful only in fixing definitely the upper limits of the tank and riser.

Perhaps the best way to illustrate the use of Mr. Johnson's formulas is to consider a general method of procedure in arriving at a design. The length of pipe line, L, is the one factor most often determined at the start. The area, A, of the pipe line may be fixed, within comparatively narrow limits, by questions of economy. Assuming that it is thus fixed, the value of c becomes a matter of judgment, depending on a knowledge of the frictional coefficient of the pipe material and the probable losses of head due to bends, etc. In fixing the elevation of the tank bottom, this should be given the highest value that could reasonably be expected, for the higher the value of c, other conditions remaining the same, the lower the surge level. A knowledge of the water-wheels and their characteristics will make it possible to estimate accurately the full load velocity, v_2.

It remains then to decide, first, for how great a sudden velocity change good regulation is desirable; and second, the amount of surge to be allowed, or the degree of stability, that is, the distance the water level should be allowed to drop beyond the final quiescent gradient level. The wide variation in load requirements, mechanical equipment, adaptability of the site for regulating works, and the value of the coefficient, c, make it impossible to formulate even a general rule for deciding these points. The design, however, should be best suited to the largest load change likely to occur during normal operation.

Adopting, then, a certain velocity change, $v_2 = v$, and the values of k, y_1, and z can be computed. Substituting them in Equation (4) and solving for F will give the correct area for the tank. This is net

tank area, and is exclusive of the area of the riser pipe. With very little further work, y_1 may be varied and its effect on the size of tank determined. Experiments with different velocity changes or other assumptions as to the probable value of c, can be tried, even a change in A, the area of the pipe line, may be found to give a more economical design. In short, by the use of the formulas, the whole field of possibilities can be investigated without needless work, and the combination adopted which is best suited to the exigencies of the case. Mr. Taylor.

As for the elevation of the tank bottom, a liberal allowance of depth below the calculated surge should be made for occasional larger load demands and for absolute insurance against lower surge levels due to a coefficient of friction higher than expected. The size of port and upper limits of the tank will be considered later under "rejected load."

To illustrate how closely the formulas may be depended on to approximate the actual performance of a design, under demanded load, four numerical cases have been selected, differing widely in all respects affecting regulation. They have been first calculated by the formulas, then by the method of arithmetic integration, and the two results compared.

The assumptions are given in Table 1.

TABLE 1.

Case.	Length of pipe line, in feet. L.	Area of pipe, in square feet. A.	Gross head, in feet. H.	Frictional coefficient. c.	Velocity, in feet per second. v_1.	Velocity, in feet per second. v_2.	Stability, or k.
S	10 000	100	250	0.2	7	10	1.7
T	6 440	200	300	0.05	6	20	2.2
U	32 200	100	1 000	0.56	7.2	9	1.15
V	3 220	113	300	0.0555	12	24	1.05

In using the formulas, it is necessary to deal entirely with velocity changes, though, fundamentally, the load changes, in terms of power, are what influence the design. To change one into the other is comparatively simple, if it is remembered that v_2 is the conduit or draft velocity at the bottom of the surge, or end of the quarter cycle. The head at that time being less, the velocity change, $v_2 - v_1$, represents more than the actual load change by an amount depending on the size of surge and the net effective head. Expressing the relation mathematically:

$(H - c\,v_1^2)v_1$ = original power before load change,

$(H - c\,v_1^2)v_0^1$ = demanded power at instant load is thrown on and before riser level has dropped,

v_0^1 being the initial draft velocity for the new load, in terms of the conduit velocity,

$(H - c\,v_1^2 - y_1)v_2$ = demanded power at the end of the quarter cycle with riser level at the bottom of the surge.

Mr. Taylor.

As the demanded power must be held constant after the initial change,

$$(H - c\,v_1{}^2)v_0{}^1 = (H - c\,v_1{}^2 - y_1)v_2.$$

Or the actual load change in horse-power can be expressed as follows:

$$\frac{A\,v_0{}^1\,(H - c\,v_1{}^2) \times 62.5 \times \text{Efficiency}}{550} - \frac{A\,v_1\,(H - c\,v_1{}^2) \times 62.5 \times \text{Efficiency}}{550}$$

and, if the water-wheel efficiency is assumed to be constant, as has been done throughout this discussion, the load change, in horse-power is

$$\frac{A\,(H - c\,v_1{}^2) \times \text{Efficiency}}{8.8}\,(v_0{}^1 - v_1).$$

The results obtained for the trial cases by Mr. Johnson's formulas, and using his nomenclature, are given in Table 2.

TABLE 2.

Case.	Z.	F, in square feet.	y_1, in feet.	t, in seconds.	a_0, in square feet.	a_1, in square feet.
S	11.65	589	17.35	79	8.97	6.49
T	28.87	2 800	40.00	89	55.2	43.5
U	9.24	709	18.82	217	5.18	2.32
V	24.43	2 038	25.2	134	33.7	9.25

The same cases were then computed by using arithmetic integration, following the changes in velocity and water levels second by second. A head had to be assumed and the velocity change at the start corresponding to the load change calculated. As the head at the quarter cycle was unknown without a trial, that given by the formulas in Table 2 was used, and the value of $v_0{}^1$ computed from $v_2 \times$ net head quarter cycle $= v_0{}^1 \times$ net head at start. The initial velocity change is then $v_0{}^1 - v_1$, and as the pressure level in the riser drops, v^1, the draft velocity, increases to a value, v_2, to hold the power constant. The resulting surge was found to be near enough to the assumed value to require no revision of the value $v_0{}^1$.

The friction and velocity head in the riser and the inertia of the water in both tank and riser were neglected, and 3 sec. were allowed for the governors to open up the turbine gates. A riser area equal to A was used, and the water-wheel efficiencies were assumed to remain constant. The work was first carried through using a port equal to a_0 and then, by trial and error, the best size of port was determined and the resulting performance calculated. The best size is one which

gives an initial drop in the riser level equal to the final level of tank and riser at the end of the quarter cycle. It must be understood that all port areas in Mr. Johnson's formulas, as well as any figures for port area appearing in this discussion, assume a 100% coefficient of discharge, so that the actual ports when built must be increased to allow for losses. Mr. Taylor.

The results are given in Table 3, together with those obtained by the formula, for the sake of comparison.

TABLE 3.

Case.	Size of port, a_0, in square feet.	Surge, y_1, with port = a_0, in feet.	Best size of port, a, in square feet.	Surge, y_1, with port = a, in feet.	Surge, y_1, by formula, in feet.	Percentage of increase of surge over result by formula.
S	8.97	18.20	8.1	17.51	17.35	0.9
T	55.2	41.49	52.6	40.52	40.00	1.3
U	5.18	20.16	4.36	19.14	18.82	1.7
V	33.7	27.29	30.0	26.45	25.2	5.0

The results obtained by formula and those computed by arithmetic integration with a port area equal to a_0, as well as the best size of port, were plotted together for comparison, and are shown in Figs. 5, 6, 7, and 8. Only the riser levels are shown, as they alone affect the regulation directly. In all cases the tank and riser levels coincide at the beginning and end of the quarter cycle.

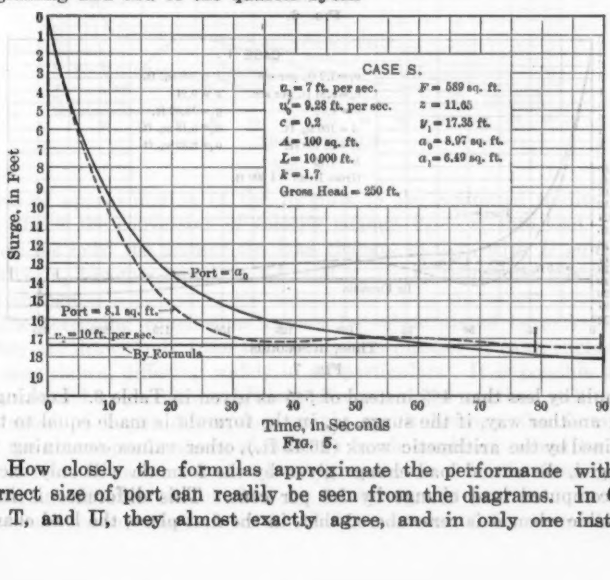


FIG. 5.

How closely the formulas approximate the performance with the correct size of port can readily be seen from the diagrams. In Cases S, T, and U, they almost exactly agree, and in only one instance,

Mr. Taylor.

Case V, do they differ by as much as 2 per cent. In computing this particular case, if the results of the formula are revised to be identical with the conditions of the arithmetic integration, making $v_2 = 24.10$ instead of 24.00, y_1 will be found to be 25.45 ft. instead of 25.2 ft., so that the computed result really only exceeds that obtained from the

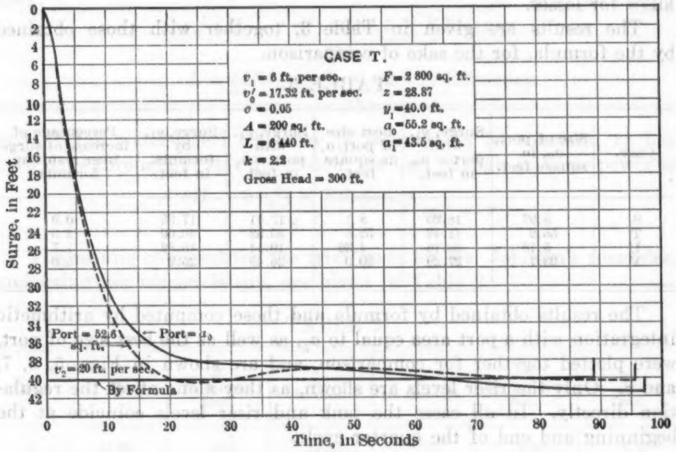


FIG. 6.

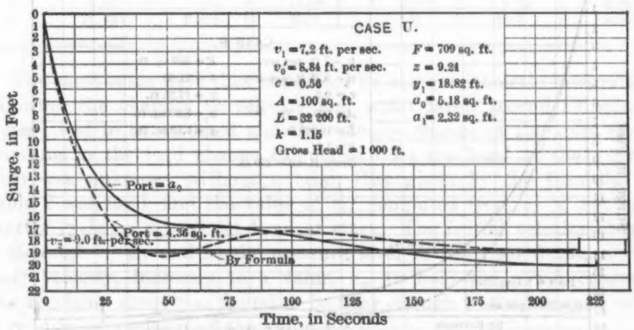


FIG. 7.

formula by less than 4% instead of 5% as given in Table 3. Looking at it in another way, if the surge, y_1, in the formula is made equal to that obtained by the arithmetic work (26.45 ft.), other values remaining unchanged, the actual load change given by the formula will only exceed the computed load change by 1.5 per cent. This difference is inappreciable when it is remembered that, in the first place, the load change

and the value of c for which a tank is designed can only be estimated. This indicates that, as we approach dead-beat stability conditions, or values of k near unity with large load changes, where the difference between the theoretical ports a_0 and a_1 is large, the formula drifts slightly away from the actual results. In the other direction, with values of k greater than 1.2 and velocity changes of 10% or more it has been found that the two methods practically coincide. The effect of assuming a larger head was tried in Case S, 650 ft. instead of 250 ft., and the resulting surge was computed as substantially the same. The value of $v_0{}^1$ computed from the same v_2 as described, however, is greater with the greater head, so that the actual load change represented by the same surge is larger in the case of larger heads. This would indicate that varying the head does not materially affect the Mr. Taylor.

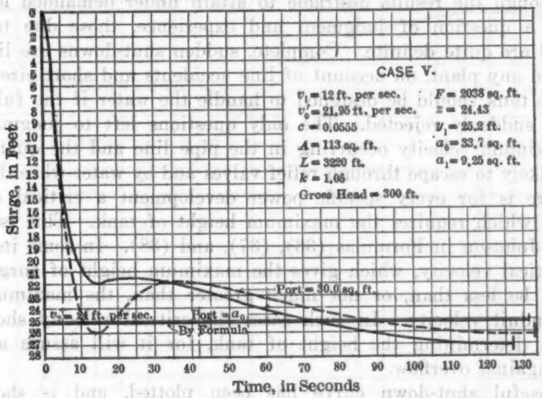


FIG. 8.

degree of approximation of the formula to the computed results, but only affects the conversion of velocity change, $v_2 - v_1$ into load. The higher the head the greater the load change, so that, with H equal to infinity, v_2 becomes equal to $v_0{}^1$, and $v_2 - v_1$ represents the actual load change.

The foregoing conclusions cannot be substantiated beyond doubt, as they are drawn only from the results of a number of cases including the ones shown, differing widely in all particulars. It is possible that, with unusual or peculiar conditions of one kind or another, still uninvestigated, a larger discrepancy might result between the formulas and the actual performance so that too much emphasis cannot be laid on the fact that after a design has been determined by the formulas it should in all cases be checked by arithmetic integration. Another point—not to be overlooked—is the efficiency curves of the water-wheels.

Mr. Taylor. If there is a considerable drop in efficiency from three-quarters gate to full gate, as is frequently the case, it means that an added draft must be taken by the wheels to overcome the loss in efficiency, and the actual surge for a certain load demand may be a great deal more than computed, enough in some cases to interfere seriously with good regulation. If the gates can be blocked back or the wheels designed so as to be at their highest efficiency when running at full gate, surge regulation becomes much easier under both demanded and rejected loads.

Retardation.—A tank having been made ample in diameter and depth for the greatest demanded load for which good regulation is desired, it is next necessary to fix the upper limits of the tank and riser in order that they may be sufficient to store the water following a sudden shut-down.

Although the results desirable to attain under demanded load are largely a question of judgment and experience, those due to loads dropped are quite definite. Complete, sudden shut-downs are likely to occur on any plant, on account of line accidents and short circuits, so that the tank should be designed to handle the water if the full plant load is suddenly rejected. The only questions left to judgment are the maximum velocity occurring in the pipe line and the quantity of water likely to escape through relief valves and by water-wheel leakage.

There is for every specific power development a critical conduit velocity which requires the maximum height of tank. This is shown by Mr. Johnson in Formulas (36), (37), and (38). In some instances this critical velocity, which gives the maximum height of surge, happens to be less than, or not much greater than, the maximum full load conduit velocity. In such cases the critical velocity should be used in determining the height of tank, for it will assure absolute safety against overflow.

A useful shut-down curve has been plotted, and is shown in Fig. 9. Having once determined the critical velocity by the formula, $v = \frac{0.0993}{c} \sqrt{\frac{A\ L}{F}}$, which is merely Formula (36) in a more simple form, the height of surge to be expected at any other conduit velocity can be picked off graphically without additional computation. The height to which the water will rise, in case there occurs a higher conduit velocity than expected, or the effect of varying the assumption as to the value of c, can be determined with quite a saving of time by the use of this curve.

The great danger to be avoided in computing shut-downs is an error in the assumption of the coefficient, c, which, if smoother than figured, will cause the water at quiescent full load to stand higher and therefore attain a higher surge level than contemplated. For this reason, it is always better, when considering rejected loads, to use the smallest value of c that it is reasonably possible to expect.

The four cases previously referred to under acceleration were also computed for shut-down. The critical velocity giving the highest surge, was so far above v_2 in Cases S, T, and V that it was disregarded as a possibility, and the surge was computed for a maximum velocity of v_2. Nevertheless, in a practical case, care should be taken to allow for a slightly higher velocity than the regular quiescent full load velocity, due to increased draft on surges. A shut-down in Case V was computed with the critical velocity of 23.9 ft. per sec., which happened to be slightly less than v_2. It will be found by trial that, provided c is not made smaller, or the area of tank or conduit changed, no variation in velocity or length of pipe line will result in a higher surge level. Mr. Taylor.

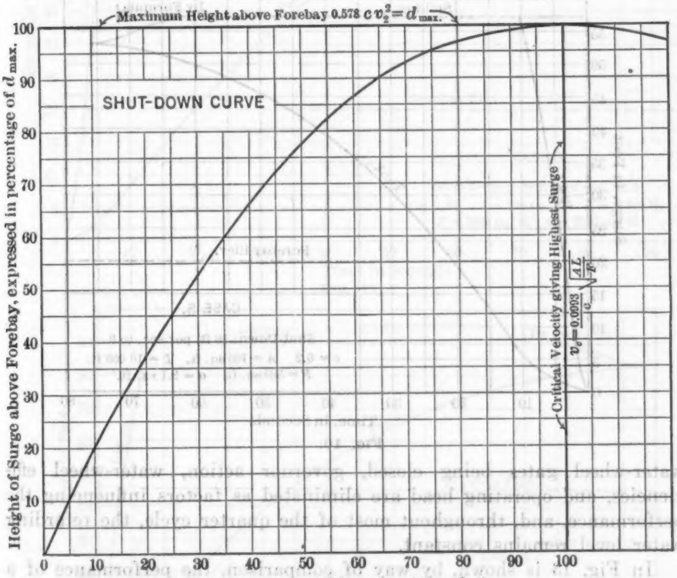


FIG. 9.

The heights of riser and tank for these four cases were first obtained by Formula (31), solving for the value of y_1 and using F as determined for demanded load. This height, y_1, was adopted for the riser, and the performance was computed by arithmetic integration. The results are shown graphically in Figs. 10, 11, 12, and 13, and the dotted line indicates the time during which the riser was spilling and represents also the height of surge as computed by the formula. The added

Mr. Taylor.

back pressure due to the weir height on the top of the riser when spilling, and the inertia and velocity head in tank and riser were neglected. Leakage through the wheels or discharge through relief valves was not considered.

In every case the mathematical formulas lean toward the side of safety, giving a somewhat higher tank and riser than necessary. What actually takes place, however, in a shut-down, agrees so nearly with the premises of the theoretical formulas that it is a question in the writer's mind whether they may not give results fully as close to what really takes place as those derived by arithmetic integration, unless the latter is computed in very small increments of time, and inertia. Velocity head and weir heights are taken into consideration. The

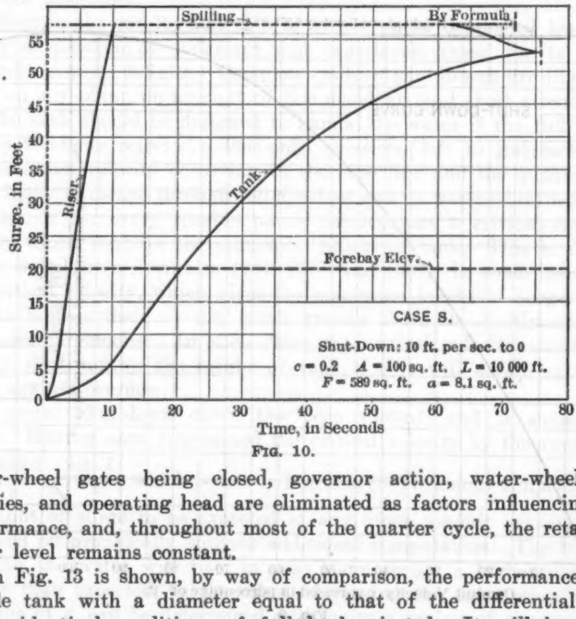


FIG. 10.

water-wheel gates being closed, governor action, water-wheel efficiencies, and operating head are eliminated as factors influencing the performance, and, throughout most of the quarter cycle, the retarding water level remains constant.

In Fig. 13 is shown, by way of comparison, the performance of a simple tank with a diameter equal to that of the differential tank under identical conditions of full load rejected. It will be noted that the surge in the simple tank attains a level twice as high above the forebay. It reaches a height of 35 ft. above, and the differential tank only touches 17 ft.

The conclusions to be drawn from the consideration of rejected loads are that the formulas very nearly approximate what actually takes place. Even more reliance can be placed on them than in the case of demanded loads. It is the writer's opinion that in the ordinary

case they could be relied on to determine the design without further checking; yet, as there is a remote possibility that with certain peculiar conditions a discrepancy might appear, it is advisable to use some form of check or allow a factor of safety in extra height. Mr. Taylor.

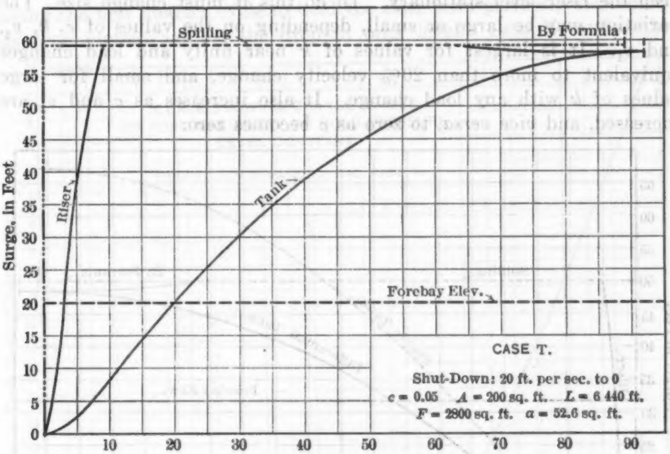


FIG. 11.

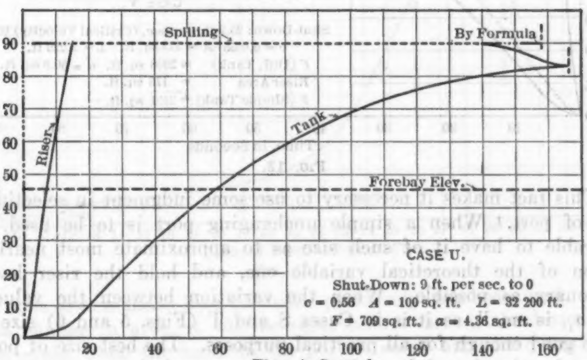


FIG. 12.

Ports.—The question of ports is very interesting as well as important, for the whole differential principle depends for its success on the choking action of the port. Pressure level changes following a load change are directly affected by the size of the port. Through it must

Mr. Taylor.

flow the water to keep the riser level from dropping too far on demanded loads or rising too high on ordinary loads rejected.

In order to treat the subject mathematically, Mr. Johnson assumes that the port just supplies or receives enough water from the tank to keep the riser level stationary. To do this it must change size. The variation may be large or small, depending on the values of c, k, v_2, and v_1. It is largest for values of k near unity and load changes equivalent to more than 20% velocity change, and small for large values of k with any load change. It also increases as c and v_2 are increased, and *vice versa,* to zero as c becomes zero.

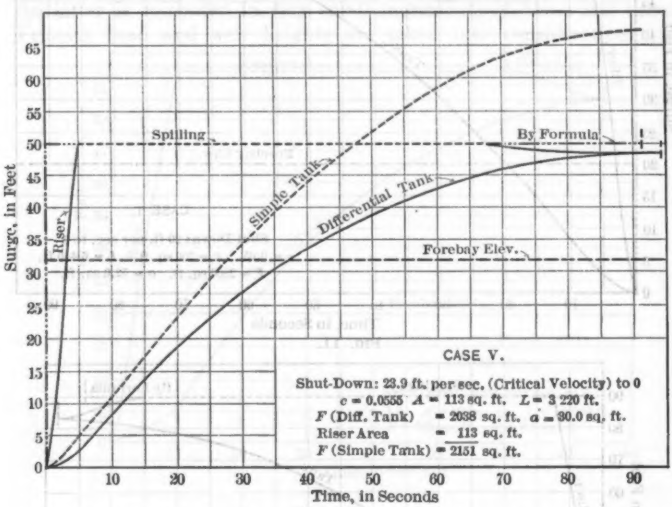


FIG. 13.

This fact makes it necessary to use some judgment in selecting the size of port. When a simple unchanging port is to be used, it is desirable to have it of such size as to approximate most nearly the action of the theoretical variable one, and hold the riser level as stationary as possible. When the variation between the values, a_0 and a_1, is small, as it is in Cases S and T (Figs. 5 and 6) size a_0 is quite good enough for all practical purposes. The best size of port, as shown by the dotted line, does not show much improvement. Even in Case V (Fig. 8) where the variation is large, there is little need for refinement unless to save a foot or so of surge. In general, however, when the variation is large, a trial by arithmetic integration should be made, and the area, a_0, should be reduced somewhat for better results. A rule that may save needless trial computation is to

assume the port area equal to $a_0 - (0.1 \text{ to } 0.3) \times (a_0 - a_1)$, using the factor 0.3 when the difference is small, varying to 0.1 for large differences. Mr. Taylor.

The reason a_0 so often cannot be much improved is illustrated by the curve shown in Fig. 14. The variation of port area necessary during the first quarter cycle to hold the riser level perfectly stationary has been plotted against the drop in tank level. Case V, under the

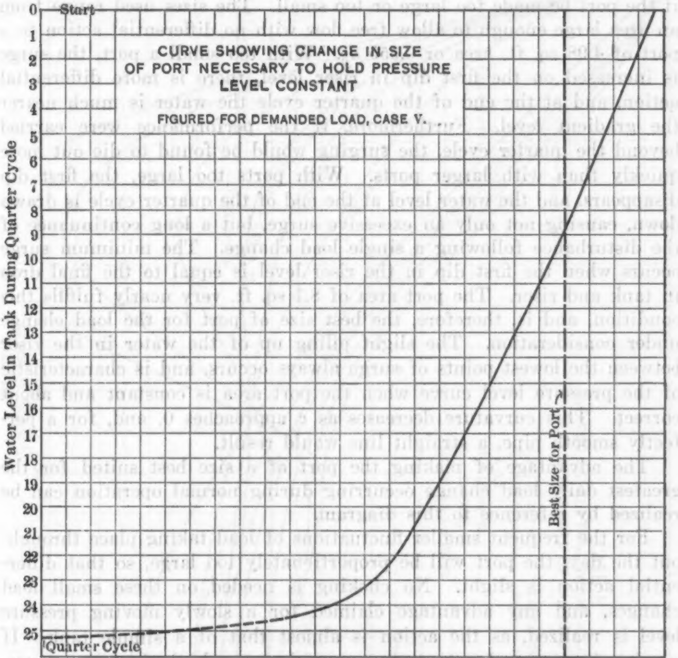


FIG. 14.

conditions of demanded load discussed previously, has been used. Most of the change in area, it will be seen, occurs during the latter part of the cycle, when the head on the port is small and the quantity of water necessary to supply the wheels from the tank has been much diminished. During this period a variation from the theory has little effect on the performance. It is important during the first half cycle, when the port is discharging large quantities of water, that its actual size should not be far from the theoretical. The curve shows that dur-

Mr. Taylor. ing this period the theoretical size does not depart to any great extent from the original area, a_0, and, for this reason, a_0 will often suffice for all practical purposes.

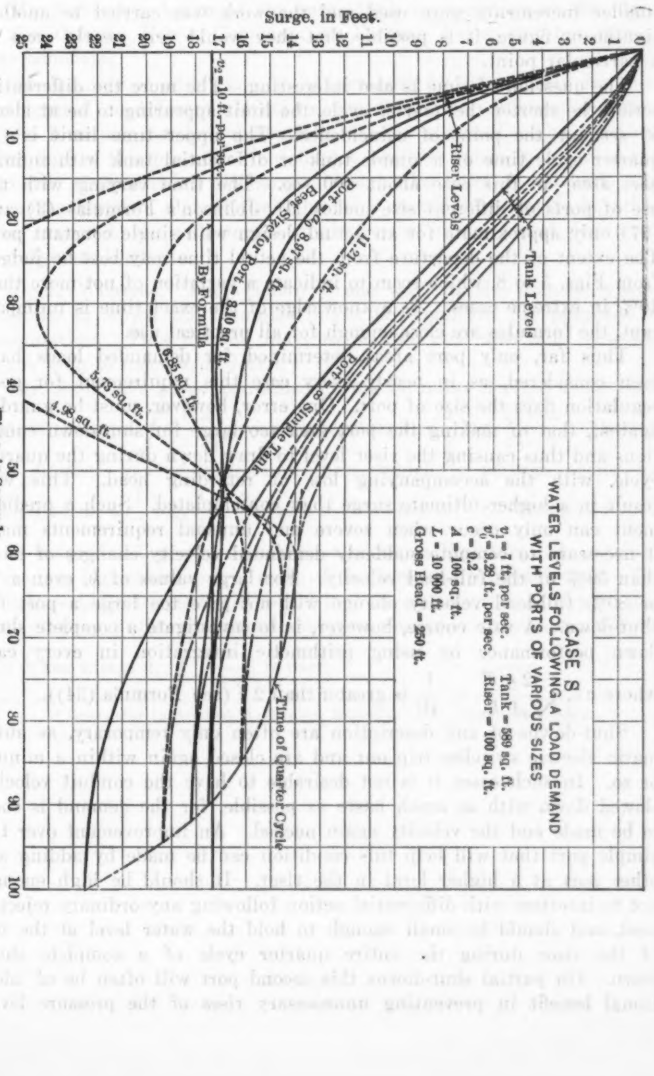
The performances obtained with the use of various sizes of ports for the same conditions of velocity and load change were computed for Case S, and are shown graphically in Fig. 15. This diagram gives a clear conception of exactly what will take place in any specific case, if the port be made too large or too small. The sizes used range from an area large enough to allow free flow with no differential action to a port of 4.98 sq. ft. area or 0.555 a_0. With too small a port, the surge is increased on the first dip in riser level, there is more differential action, and at the end of the quarter cycle the water is much nearer the gradient level. Furthermore, if the performance were carried beyond the quarter cycle, the surging would be found to die out more quickly than with larger ports. With ports too large, the first dip disappears, and the water level at the end of the quarter cycle is drawn down, causing not only an excessive surge, but a long continuance of the disturbance following a single load change. The minimum surge occurs when the first dip in the riser level is equal to the final drop in tank and riser. The port area of 8.1 sq. ft. very nearly fulfills this condition, and is, therefore, the best size of port for the load change under consideration. The slight piling up of the water in the riser between the lowest points of surge always occurs, and is characteristic of the pressure level curve when the port area is constant and about correct. This curvature decreases as c approaches 0, and, for a perfectly smooth pipe, a straight line would result.

The advantage of making the port of a size best suited for the greatest daily load change occurring during normal operation can be realized by reference to this diagram.

For the frequent smaller fluctuations of load taking place throughout the day, the port will be proportionately too large, so that differential action is slight. No choking is needed on these small load changes, and any advantage claimed for a slowly moving pressure level is realized, as the action is almost that of a simple tank. If surging increases, due to large or synchronous load changes, as previously explained, the choking action of the port is rapidly accentuated and stops any inclination of the surge to increase beyond bounds. For greater load changes than the normal maximum daily which may occasionally occur, the port is proportionately too small, and from the diagram it will be seen that the surge is quickly dampened and quiescent conditions are restored.

A rather remarkable and interesting coincidence to be noted on the diagram is the fact that, no matter what size the port, after 53 sec. the water stands at approximately the same level in the riser, and it does this whether the water is rising or falling at that time. What

Mr. Taylor.



Time, in Seconds

FIG. 15.

Mr. Taylor. significance attaches to this fact and whether theoretically the curves should all meet in a point, the writer has been unable to determine. They were computed by arithmetic integration, using at the start time intervals of 1 sec., and increasing to 2 sec. after about 20 sec. If smaller increments were used and the work was carried to another significant figure, it is possible that they would very nearly cross at a particular point.

The question of time is also interesting. The more the differential action the shorter the quarter cycle, the limit appearing to be at about 53 sec., or the point of coincidence. The upper time limit is the quarter cycle time of a simple tank or differential tank with infinite port area—in this case about 100 sec. The time varying with the use of ports of different size makes Mr. Johnson's Formulas (3) and (27) only approximate for an actual design with single constant port. The extent of the departure from the actual time may best be judged from Figs. 5 to 8, which seem to indicate a variation of not more than 10% in extreme cases. As a knowledge of the exact time is unimportant, the formulas are close enough for all practical use.

Thus far, only port areas determined for demanded loads have been considered, as in nearly every case this requirement for good regulation fixes the size of port. One error, however, must be guarded against, that of making the port area too large for shut-down conditions and thus causing the riser level to draw down during the quarter cycle, with the accompanying loss of retarding head. This will result in a higher ultimate surge than contemplated. Such a predicament can only occur when severe and unusual requirements make it necessary to compute suddenly demanded velocity changes of more than 50% of the full-load velocity. For large values of k, even a 70 or 80% full-load velocity change will not give too large a port for shut-down. A safe course, however, is to investigate a complete shut-down performance by using arithmetic integration in every case where $c\,v_2 \sqrt{\frac{2\,g\,F}{A\,L}} + \frac{1}{10}$ is greater than $2\,r$ (see Formula (34)).

Shut-downs of any description are often only temporary, as automatic electric switches trip out and are closed again within a minute or so. In such cases it is not desirable to have the conduit velocity slowed down with as much haste as possible, for the demand is soon to be made and the velocity again needed. An improvement over the simple port that will help this condition can be made by adding another port at a higher level in the riser. It should be high enough not to interfere with differential action following any ordinary rejected load, and should be small enough to hold the water level at the top of the riser during the entire quarter cycle of a complete shut-down. On partial shut-downs this second port will often be of additional benefit in preventing unnecessary rises of the pressure level.

and a series of small ports around the riser will not only reduce the volume of water spilling over the top, but cushion what does spill against pounding down into the tank, by traversing the falling water with the spouting streams from the ports. This device is a feature of Mr. Johnson's design of the surge tank in operation at the plant of the Ontario Power Company, at Niagara Falls, and has proved very beneficial. Mr. Taylor.

The number of ports can also be increased, and if properly designed and located at different elevations, the theoretical differential action of the formulas can be substantially exactly obtained. The practical advantage of this procedure is doubtful, however, when it is considered that the coefficient of discharge through the holes is at best only an approximation, and, if the value of c should turn out to be other than expected, the refinements might result in an actual detriment to differential action.

D. L. WEBSTER,* ESQ.—There is one way, in Mr. Johnson's conclusions, in which the formulas might be simplified for purposes of computation. This method may be pointed out, although it would be too long to discuss in detail. Mr. Webster.

Mr. Johnson's paper contains several formulas derived from integrals of the type, $\int \frac{d\,x}{1-x^2}$, which integrates into $\frac{1}{2}\log.\frac{1+x}{1-x}$. There is also another way of expressing these, which is perhaps easier for computation, as $\frac{1}{2}\log.\frac{1+x}{1-x}$ is also equal to $\tan.h^{-1}x$. This anti-hyperbolic tangent will occur in such expressions as Equation (1), in which there is a logarithm, and in most of the formulas following that. The anti-hyperbolic tangent will occur here in exactly the same way that the ordinary anti-trigonometric tangent occurs in Equation (27), in which the integral, $\int \frac{d\,x}{1+x^2}$, is used.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1325

EXTERNAL CORROSION OF CAST-IRON PIPE*

By Marshall R. Pugh, M. Am. Soc. C. E.

With Discussion by Messrs. C. P. Bowie, Kenneth Allen, William W. Brush, Samuel Tobias Wagner, R. C. Kellogg, A. D. Flinn, William J. Boucher, George M. Purver, W. J. E. Binnie, Leonard S. Doten, and Marshall R. Pugh.

Synopsis.

Cast-iron pipe is in such general use that a knowledge of its strong points, its limitations, and the precautions necessary to insure its durability under certain conditions, is of the highest importance to the engineer.

The purpose of this paper is to warn the engineer of conditions where special protection is needed against external corrosion, and to suggest preventive measures adapted to different cases, in the hope of bringing out more information in the discussion.

Tuberculation and electrolysis from stray currents have been studied elsewhere at length and will not be discussed herein.

Examples of the great durability of cast-iron pipe are first given, after which other instances are cited in which it has deteriorated rapidly. Mention of the composition of cast iron and the theories of its corrosion is followed by a study of conditions contributing to, and inhibiting, corrosion.

* Presented at the meeting of October 7th, 1914.

In the light of this information, reasons are sought for the deterioration observed in typical cases; and then precautionary and preventive measures are considered.

INTRODUCTORY.

Records of the facts relating to the corrosion of cast-iron pipe are so scattered (frequently consisting of incidental references in articles on unrelated branches of engineering), and so much of the literature is found only in publications familiar chiefly to the chemist and physicist, that, abandoning any effort at originality, it seemed worth while to gather these scattered facts and what little is known, in the hope that additional information might be brought out and light thrown on many perplexing problems; and that the whole might be placed before the engineer in accessible form.

"There is no limit to the life of cast-iron pipe", says a recent advertisement in a technical journal, and this undoubtedly voices the opinion of many well-informed persons. Opposed to that we find the following: "The relatively high rate of self-corrosion [as distinguished from electrolytic corrosion] of cast iron as compared to the other kinds of iron tested is contrary to the generally accepted idea that cast iron is more resistant to self-corrosion than wrought iron."*

This paper will be confined entirely to a study of the external corrosion of cast-iron pipe, though, obviously, many facts must be applicable as well to internal corrosion. As stated, electrolysis from stray currents has already been dealt with so fully that it will not be taken up. Tuberculation has been studied exhaustively,† and internal corrosion and hot water troubles have been investigated very ably.‡

The ancients were unacquainted with pipe of this material, as the art of casting iron was unknown until the 14th or 15th Century. The Romans, in the time of the Cæsars, used pipes of stone, bronze, and lead. Fig. 1 shows a Roman, bored stone pipe, and Fig. 2 a lead pipe, with a section the actual size of the pipe. These pipes were always oval in shape, and were made of a sheet of lead bent up and the edges soldered together. Sextus Julius Frontinus, Water

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* "Electrolytic Corrosion of Iron in Soils." by Burton McCollum and K. H. Logan, Technologic Papers, U. S. Bureau of Standards, No. 25.

† "Tuberculation and the Flow of Water in Pipes", by Nicholas S. Hill, Jr., *Proceedings*, Am. Water Works Assoc., 1907, p. 303.

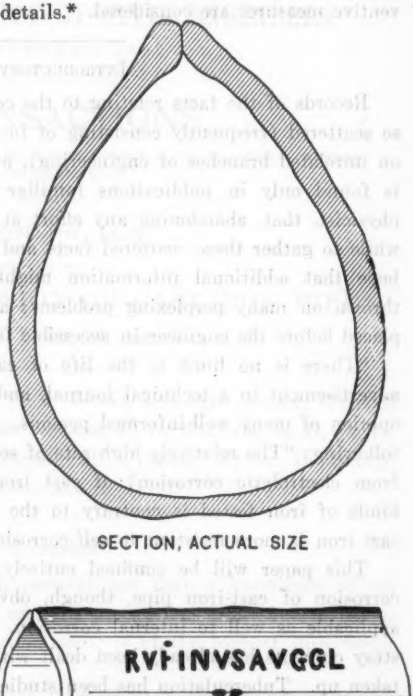
‡ "Hot Water Troubles", by George C. Whipple, M. Am. Soc. C. E., *Proceedings*, Am Water Works Assoc., 1911, p. 231.

Commissioner of Rome (*Curator Aquarum*) from 97 A. D. to 106 A. D., wrote two books describing the works under his charge, which are full of the most interesting details.*

In more modern times, London, in 1236, was provided with water from Tyburn, which was brought to the city through a lead pipe. The water rights were purchased by the magistrates from Gilbert Sandford.

ANCIENT ROMAN
BORED STONE PIPE

FIG. 1.



ANCIENT ROMAN PIPE
MADE FROM SHEET LEAD

FIG. 2.

EXAMPLES OF DURABILITY OF CAST-IRON PIPE.

We cannot go back so far in our study of the life of cast-iron pipe. The earliest records which appear to be authentic relate to several old pipes of various diameters laid, by order of Louis XIV, near

* "The Two Books on the Water Supply of the City of Rome, by Sextus Julius Frontinus", translated by Clemens Herschel, M. Am. Soc. C. E., Boston, Dana, Estes & Company, 1899; Revised and Corrected Edition, Longmans, Green & Company, New York and London.

Paris, from the reservoirs of Picardie to those of Montbauron, together with the spring water conduit, the whole supplying the town and parks of Versailles. According to the Ministry of Public Instruction and Art,* these were laid between 1664 and 1688, or from 226 to 250 years ago. They aggregate about 8 000 m. (26 000 ft.) in length, and are still in use. The great fountains at Versailles, one of which, the fountain of Neptune, is shown in Fig. 10, are supplied with water through three pipes, 20 in. in diameter, which bring the water from the Park of the Trappists to the reservoirs of Montbauron; two 20-in. and three 13½-in. pipes leading from the reservoirs to the gate-house; and two 20-in. pipes and a 13½-in. pipe leading from the reservoirs of Gobert to the reservoirs of l'Aile. These aggregate 27 500 ft. in length, were laid in 1685, 229 years ago, and are still in use. Two years later, or in 1687, the conduit of Chevreloup, 13½ in. in diameter and 11 500 ft. long, was laid from the reservoir l'Aile to the Trianon.

"All of these pipe lines consist of pipes one meter in length, joined by means of bolted flanges. They are of considerable weight, and still serve their purpose satisfactorily.

"The few repairs which have been required have generally been necessitated by the bad condition of the flange bolts, which have rusted out."*

At Clermont-Ferrand, Department of Puy-de-Dôme, on September 14th, 1746, the Commissioners of Fountains made an agreement with a Parisian iron merchant, Sieur Marchais, for furnishing about a mile of 5 or 6-in., cast-iron pipe to supply water to the fountains. This pipe was laid in 1747-48. The lower portion, in the plain, was under quite heavy pressure, and was $\frac{8}{10}$ in. thick; where the pressure was less, the thickness was only ½ in. These pipes were in 3-ft. lengths, flanged, and with a lead gasket sometimes as thick as ⅜ in. In 1867, the system was paralleled by new mains, but much of the original pipe is still in use.

At Saint Etienne, in 1904, there were cast-iron pipes which had been in service since 1782, or 122 years.*

A few years ago, the City of Paris* abandoned the Chaillot Pumping Station, the first erected for supplying that city with water. The

* "The Life of Cast-Iron Pipe", by C. Cavallier, *Journal*, New England Water Works Assoc., June, 1904. p. 218.

intake, consisting of flanged cast-iron pipe extending out into the Seine, was removed after having been immersed in the river for more than a century. When removed the pipes were found to be so well preserved that they could readily have been used again, except for the fact that they were of an antique and obsolete pattern. After removing the scale and shellfish coating from the exterior, the inscription, "CREUSOT, AN 10", in raised letters, could be readily seen. This date corresponds to the year 1802 in the present calendar.

The metal was a gray, cast iron of excellent quality. No alterations of either the interior or exterior appeared after its 100 years of service, and the fractures were as sharp as those of a fresh casting.

London and Glasgow have a record of 120 years of service for cast-iron pipe. The first extensive works in the former city were built by Sir Hugh Middleton between 1609 and 1613. As early as 1746, the Chelsea Water Company, of London, laid a 12-in. flanged, cast-iron pipe, which was relaid in 1791 on account of the joints "being perished." It was the engineer of that water company, Mr. Thomas Simpson, who designed the first bell and spigot pipes. This was about 1785, at which time an experimental section with lead joints was laid.*

As a rule, however, London was supplied with water through wooden pipes until about 1820.

In the United States, the earliest pipes were of bored logs. Philadelphia adopted them, although, in 1804, a length of about ¾ mile of iron pipe was laid as an experiment. The new Fairmount Water-Works in that city had a distribution system of cast-iron pipes, which was the first in America, at all events on an extensive scale.

These pipes were imported from England, and were of the bell and spigot type, in 9-ft. lengths. A curious feature about them was that each length had three or four ribs or rings about it, approximately 1½ in. wide, which were presumably to afford places with heavier walls in which to tap the pipe.† These pipes are of particular interest owing to the fact that they are still in use after 95 years, with every evidence of continuing to give a good account of them-

* "The Development of Water Supplies and Water-Supply Engineering," by Frederic P. Stearns, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LVI, p. 455.

† Information regarding Philadelphia water pipes was furnished by the Bureau of Water of that city.

selves. Though forming only an infinitesimal portion of the present distribution system, they, nevertheless, constitute quite a considerable mileage.

A few of these pipes are noted in order to give an idea of their character: A 4½-in. cast-iron pipe on Chestnut Street, from Broad to 15th Street, laid in 1817, was removed only recently. A 22-in., cast-iron main leads from Fairmount Basin to Green Street, to Pennsylvania Avenue, to Callowhill Street. It then reduces to a 20-in. main leading to Broad Street and thence to Chestnut Street. This pipe was laid in 1819, and, with the exception of a portion which was removed in 1897 on account of the construction of the Pennsylvania Avenue Subway, is still in use. Mains on Market Street, from Juniper to Broad Streets, laid in 1820, from Eleventh Street to Juniper Street, laid in 1823, and from Water Street to Eleventh Street, laid in 1822, were in good condition when removed in 1907 to make way for the Market Street Subway. Mains, 6 and 10 in. in diameter, on Second Street, between Greenwich Street and Lehigh Avenue, were laid between 1823 and 1852; 8-in. mains on Front Street, between Wharton Street and Laurel Street, were laid at various dates between 1822 and 1834; one 10-in. main on Chestnut Street, from Front Street to Broad Street, was laid in 1821 and 1823; and a 20-in. main on Fifteenth Street, from Callowhill Street to Chestnut Street, was laid in 1829. All these mains are still in use.

This list might be extended to several pages, but it serves to show that a century's use has not rendered these pipes unfit for service. Virtually the whole of the old distribution system laid at that time has given an equally good account of itself.

The late James B. Francis,* Past-President, Am. Soc. C. E., cited the case of Lowell, Mass., as follows:

"It is within my experience and knowledge that pipes at Lowell, which were obtained from Philadelphia in 1830, were then intended to be used under a pressure of 150 feet head, and are now in use under 200 feet head, without any signs of failure after sixty years' use."

Even when immersed in the fresh water of streams and rivers, this resistance to corrosion is manifested. Cast-iron cannon taken

* *Transactions*, Am. Soc. C. E., Vol. XXIV, p. 261.

from a vessel sunk in the Delaware River for more than 40 years were perfectly free from rust.

Summing up what has been observed, it may be said that our experience of 250 years with cast-iron pipe has not been sufficiently long to establish just what its life is.

Examples of Deterioration of Cast-Iron Pipe.

A knowledge of this fact, however, may lead to very mistaken conclusions. One of the earliest observations relating to the rapid deterioration of cast-iron pipe was that of Mr. Thomas Duncan, who, in 1853, in describing the Liverpool Corporation Water-Works, said:

"During the process of taking up and adjusting the pipes in the lower districts, near the margin of the docks, where the soil is impregnated with muriate [common salt] they were found, in many instances, to have become so soft, that they could easily be cut by a knife. This was found to be the case where they had not been laid for more than twenty years; whereas, in the higher districts, where they had been laid for nearly fifty years, they were found to be as good as when first laid down. In all instances the hardest pipes had deteriorated least, and were found cleanest on the inside."*

In other words, under certain conditions, cast-iron pipe is not durable, unless protected. Therefore, it is of great importance to the engineer to determine just what those conditions are, and how to guard against them.

The salt of the ocean is particularly destructive to cast iron, and its action is manifested in a peculiar way, differing materially from ordinary rusting, scaling, and pitting.

In contrast to the case in which cast-iron cannon removed from the fresh waters of the Delaware River were found to be free from rust, Major-General Pasley† stated that the cannon of the *Royal George,* which had been sunk in the ocean for 62 years, and of the *Edgar,* which had been similarly submerged for 133 years, had been examined by him. The cast iron had generally become so soft that the metal, which resembled plumbago in appearance, could be readily cut with a knife. The cast-iron shot became red hot on exposure to air, and fell to pieces. The cannon hardened after a time, and the late William J. McAlpine,‡ Past-President, Am. Soc. C. E., said

* *Minutes of Proceedings*, Inst. C. E., Vol. XII, p. 487.

† *Minutes of Proceedings*, Inst. C. E., Vol. III, p. 86.

‡ *Transactions*, Am. Soc. C. E., Vol. I, p. 23.

that they were again fired. Be that as it may, the cast-iron guns from some ancient pirate ship were brought up from the ocean depths off Holyhead in 1822, after the lapse of a century. They were quite soft, but hardened so much on exposure to the air that they were used to fire salutes to King George IV, when he passed through Holyhead on his way to Dublin.* These old guns were said to have given louder reports than any others!

The chemist, Berzelius,† also refers to the curious heating of cannon balls, and states that some which were raised at Karlskrona, from a ship that had been sunk for 50 years, were found to have been converted into a porous, graphitic mass which heated spontaneously on exposure to air for 15 min.

It is well known that iron filings will burn in the flame of a candle, and that very finely divided iron, obtained by the reduction of its oxides in hydrogen, is pyrophorous, that is to say, it ignites spontaneously on exposure to air. These cannon balls, therefore, may have contained a remnant of metallic iron in an exceedingly fine state of subdivision. The heating may also have been due to the rapid change from ferrous to ferric hydrate on exposure to air, or to a combination of the two causes. Just why the shot should heat up and fall to pieces, when the cannon themselves did not, is difficult to explain.

The corrosive action of sea water is not uniform under all conditions, but may be materially accelerated or retarded. A case of this sort is cited by Mr. Robert Mallet:‡

"Iron under certain circumstances is subject to a peculiar increase of corrosive action—as, for instance, cast-iron piling at the mouth of tidal rivers—from the following cause. The salt water being of greater density than the fresh, forms at certain times of tide an under current, while the upper or surface water is fresh; these two strata of different constitution coming in contact with the metal, a voltaic pile of one solid and two fluid elements is formed; one portion of the metal will be in a positive state of electrical action with respect to the other, and the corrosive action on the former portion is augmented. The lower end of an iron pile, for instance, under the circumstances just mentioned, will be positive with respect to the other,

* Rennie, *Minutes of Proceedings*, Inst. C. E., Vol. IV, p. 333.
† Traité de Chimie, 1831, Vol. III.
‡ *Minutes of Proceedings*, Inst. C. E., 1840, p. 71.

and the corrosion of the lower part will be augmented by the negative state of the upper portion, while the upper will be *itself* preserved in the same proportion."

This statement would seem to be borne out by observations on the Brandywine Lighthouse,* situated on a shoal of that name in Delaware Bay, 8 miles inside the Capes. It is founded on nine, hammered-iron, screw-piles, surrounded by fifty-two rolled-iron, fender-piles. It was built in 1849-50, in about 6 ft. of water, but at times the scour and shifting sands have varied this depth materially. In 1873, an examination by a diver revealed the fact that the most serious corrosion was below the surface of low water, beyond the action of the atmosphere. In 1879, one of the fender-piles, together with a portion of the cast-iron socket, was broken off by ice. The outer ⅛ in. of this casting consisted

"of a soft dark gray coating of metamorphosed iron, which could be removed readily with the finger-nail, but the piece retained the sharp outlines of its original form. The coating became hard after exposure to the air and did not separate from the iron, but the iron seemed rather to have become softened by the action of the water on its surface. * * *

"No greater deterioration, where the wrought and cast iron were in contact, was observable."

Cases occur, and with considerable frequency, where trouble has resulted from laying cast-iron mains in saline soils.

In 1902, the Elizabethtown Water Company, Elizabeth, N. J., laid about 2 miles of 6-in., cast-iron main across salt meadows. It was ordinary coated water pipe, about 0.45 in. thick, and was laid about 8 in. below the surface of a salt marsh.† The surface of the marsh was about 18 in. above ordinary high water, and, therefore, was covered at times by salt water. The nearest trolley is about 3 miles away, so that stray currents have nothing to do with the observed deterioration of the pipe. In 1909, or only 7 years after it was laid, the pipe began breaking, under a pressure of 40 lb., and it became necessary to renew the whole line in 1912, after a life of only 10 years.

Internally, the pipe was in good condition. The external appearance was equally good, there being an entire absence of pitting. On

* *Proceedings*, Engrs. Club of Philadelphia, 1884, p. 129.

† Information furnished by Mr. D. H. Townley, Superintendent, Elizabethtown Water Company.

closer examination this appearance was found to be deceptive, for, in many places, sometimes the top, sometimes the bottom, and, at other times, the sides, were found to have large spots where the iron appeared to be leached out, so that it could be cut like putty, or rather like soft graphite, which it strongly resembled in appearance. This action was not regular, sometimes skipping several feet, but was worst at the bottom of the pipe. In some instances, these soft places had been eaten completely through the walls of the pipe, which, although not pitted or swelled out, had entirely lost all the characteristics of iron. After exposure to the air, this metamorphosed iron hardened, just like the cannon used in saluting King George IV at Holyhead.

At Perth Amboy, N. J., similar trouble was experienced. A 16-in. main, crossing a salt meadow for some 3 500 ft., had to be abandoned altogether in less than 20 years. Mr. A. H. Crowell, Superintendent of the Water-Works, under date of March 20th, 1913, says:

"Our second main, a 24-in., laid under the same conditions [as the 16-in. main] about seven years ago, now shows evidence of the same deterioration, softening in spots so that it may be pared with a knife on the exterior. This line the Board has decided to uncover and leave exposed in open trench to the flow of the tides, our experience having shown that pipe when so laid exposed to the light and air is not affected; in other words, we have reached the conclusion that this particular deterioration occurs from chemical action due to contact of roots of salt grasses.

"In connection with the deterioration, one peculiarity noted is that when such pipe is removed and exposed to the light and air for a period of several weeks it will harden and ring apparently sound."

This pipe, which was laid in black meadow muck from which salt grass is cut, is buried with 3 ft. of cover. The meadow is completely submerged at extreme tides, with an ordinary range of about 4 ft. The pipe was ordinary gray cast iron, tar-coated, weighing 140 lb. per ft., and the deterioration occurred at all points, but principally on the top and upper sides. As in the Elizabeth pipe, the exterior did not pit, but deterioration occurred from softening in spots which finally blew out.

At Atlantic City, N. J.,* the water supply is delivered to the city from the Absecon Pumping Station, the mains leading through salt

* Information furnished by Mr. Lincoln Van Gilder, Engineer and Superintendent, Water Department.

marsh for a distance of 22 000 ft. In 1882, a 12-in. cast-iron main was laid, which, in 1888, was paralleled by a 20-in., cast-iron main; and, in 1901, the capacity was further augmented by laying a 30-in. steel main alongside the others. In 1911, a 48-in., wood stave main was laid parallel to the preceding three for about 7 000 ft., after which it takes a different route for the remainder of the line. Both the cast-iron mains are soft in spots, and blow out from time to time in places, under a normal pressure of 50 lb. The deterioration is precisely like that noted at Elizabeth and at Perth Amboy. The steel pipe is even less satisfactory. Though laid at a later date than the two cast-iron mains, it is rapidly deteriorating, and 6 000 ft. of it are now out of service.

The top of each of the two cast-iron mains is about 1 ft. below the surface of the meadow; the steel main is about half its diameter below the surface, and the top is covered with meadow sod.

At Richmond, Va.,* about 400 ft. of 6-in., cast-iron pipe, laid in a salt blue marl, in 20 years, became soft enough to cut with a knife, the action appearing to be equally great at its top, bottom, and sides. A trolley track, 450 ft. distant, may possibly have contributed to the disintegration.

Trouble has been experienced also from saline soils at points remote from the ocean. At Syracuse, N. Y., there are salt springs which have been used in the manufacture of salt since their discovery by the Jesuits in 1654. They are situated along the shores of Onondaga Lake, and, in 1797, were taken over by the State, which passed laws governing the manufacture of salt.

The Bureau of Water† of Syracuse has had much trouble from the corrosion and disintegration of water pipe in the salt lands. A line of pipe runs in a very low section of these lands, the major part of which is under water a large portion of the year. The part which is continually submerged, however, has not disintegrated, but that adjacent to it, where the main was laid about 1904, began to fail after several years' service. The corrosive action appears to have been similar to that noted heretofore. The pipe was sound in parts, and at the leaks it could be cut with a knife. The salt has been for a century at the point where the disintegration occurs.

* Information furnished by Mr. E. E. Davis, Superintendent, Water-Works.
† Information furnished by Mr. George A. Glynn, Superintendent.

A third soil destructive to cast-iron pipes is also noted by Mr. Glynn as occurring at Syracuse. He says, in a letter:

"Since submitting this matter to you and Dr. Pattee, I have observed two cases of disintegration in differents parts of the town, the pipe being eaten precisely as that in Spring Street. In each case the soil above the pipe was composed in the main of coal ashes and apparently the constant trickle of water had created some chemical action which ate the pipe. As a layman, I have concluded that coal ashes, where there is a possibility of a flow or trickle of water through them, are responsible for this particular kind of disintegration."

A. A. Reimer,* Assoc. M. Am. Soc. C. E., notes an instance in his experience where an 8-in. main, laid in clay and ashes—marl with some cinders mixed with it—went out of service every 6 months.

In Germany, R. Krzizan† cites a case where a portion of an asphaltum-coated, cast-iron, water main, which had been in service for 20 years, suddenly became defective. A number of conical holes were scattered irregularly over the surface and were surrounded by graphite-like material containing particles of metallic iron. The composition of this corroded material is given in Table 1.

TABLE 1.—Analysis of Corroded German Water Pipe.

Metallic iron	5.98
Ferric oxide and hydroxide	34.09
Ferrous phosphate	9.63
Ferrous silicate	37.16
Ferrous sulphate	0.47
Carbon	11.42
Sulphur	0.097
Manganese	1.362
Copper	0.296
	100.505

There was nothing in the water running through the pipe to account for this formation, but crystals of gypsum were distributed irregularly in the clay in which the pipe was laid. Krzizan attributes the corrosion to local currents set up by contact between the graphite particles and the iron, in the presence of a solution of calcium sulphate at the points where the crystals of gypsum were contained in the clay adjacent to the pipe.

* *Proceedings*, Am. Water Works Assoc., 1910, p. 263.
† *Zeitschrift* für öffentliche Chemie, Vol. XVIII, p. 433.

A question arises as to what effect the alkali soils of the West would have on cast-iron pipe, and it is hoped that information on this point may be brought out in the discussion of this paper. In the effort to obtain some facts bearing on this phase of the subject, the soil survey of the Uncompahgre Valley, in Colorado, made by Messrs. Nelson and Kolbe for the United States Department of Agriculture in 1910, indicated that Montrose, on the banks of the Uncompahgre River, had areas of such high alkalinity that experience there would be useful in giving an idea of what might be expected in such cases.

"The source of most of the alkali of this area is the underlying shales, and wherever they are near the surface its presence is assured."*

The low valley soils are also in general heavily charged with alkali, the principal salts being sodium sulphate (Na_2SO_4); sodium chloride ($NaCl$), or common salt; calcium chloride ($CaCl_2$); magnesium chloride ($MgCl_2$); magnesium sulphate ($MgSO_4$); and sodium carbonate (Na_2CO_3), named in the order of the quantities in which they occur. An analysis of a typical alkali crust found on the surface of the soil is given in Table 2.*

TABLE 2.—ANALYSIS OF ALKALI CRUST.

Constituent.	Parts per 100 000.	Constituent.	Parts per 100 000.
Calcium (Ca)	Trace.	Chlorides (Cl)	2 280
Magnesium (Mg)	....	Sulphates (SO_4)	8 320
Sodium (Na)	2 870	Bicarbonates (HCO_3)	2 120
Potassium (K)	250	Carbonates (CO_3)	1 160
			17 110

The greater part of Montrose is underlaid by soil having an alkali content of from 200 to 400 parts per 100 000. There is a small area at its southwest corner where the alkali runs up to 600 parts per 100 000, and another area in the northwest portion where it ranges from 600 to 1 000 parts per 100 000.

Mr. R. L. Smith, Superintendent of Water Supply, states that the system, which was built in 1888, consisted of 4, 6, and 8-in., wrought-iron pipe, which deteriorated so rapidly that it was all replaced either by cast-iron or wood stave pipe, except about 800 ft.

* "Soil Survey of the Uncompahgre Valley Area, Colorado," Nelson and Kolbe, Washington, 1912.

of the 8-in. pipe in the southwestern part, which is still in very fair condition. Cast-iron pipe which has been in use for nearly 20 years has appeared to be in an excellent state of preservation, wherever it has been examined, and has never given any trouble. The alkali soil, however, has been disastrous to the wire of the spiral-wound wood stave pipe. It was laid in 1905 and will have to be removed within the next couple of years. There are no trolleys, and the pipe is laid 4 ft. below the surface.

Before attempting to draw any conclusions from the facts presented, it will be desirable to consider first the composition and character of cast iron, and second, to review as briefly as possible the theory of its corrosion, and study some experimental data bearing on the subject.

The Composition of Cast Iron.

Cast iron is a substance of varied composition, both physically and chemically, so that some conception of its formation is desirable in considering its corrosion.

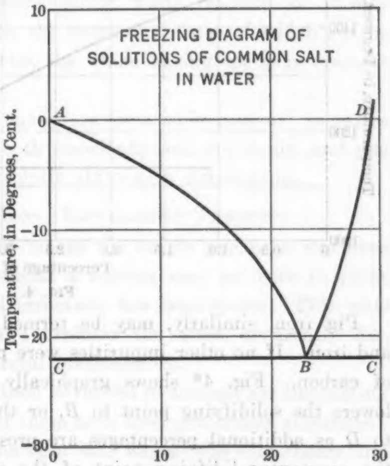


Fig. 3.

It is well known that common salt added to water progressively lowers its freezing point, until, with 23.5% of salt present, the whole mass freezes at −22° cent. This is said to be the "eutectic" composition, a term which is used of a compound substance that has a lower fusing point (or freezing point) than its components themselves. The freezing point is raised if either more or less salt is present.

The whole is shown diagrammatically by Fig. 3.* In percentages less than the eutectic, crystals of ice form when the temperature falls to the points shown on the curve, *A-B*, the mother liquor be-

* "Cast Iron in the Light of Recent Research," Hatfield.

coming more concentrated, until the eutectic composition is reached, at which time the whole solidifies. This always occurs at a temperature of — 22° cent. If the original solution contains more than 23.5% of salt, the solution progressively freezes at the temperatures shown on the curve, *B-D*, pure salt being deposited and the mother liquor becoming less concentrated, until at — 22° cent., it diminishes in saline content to the eutectic composition, when, again, simultaneous freezing takes place. The line, *C-C*, shows the freezing point of the mother liquor. Under the microscope, it is seen that the ice is not homogeneous, but consists of innumerable plates of salt and ice in mechanical contact.

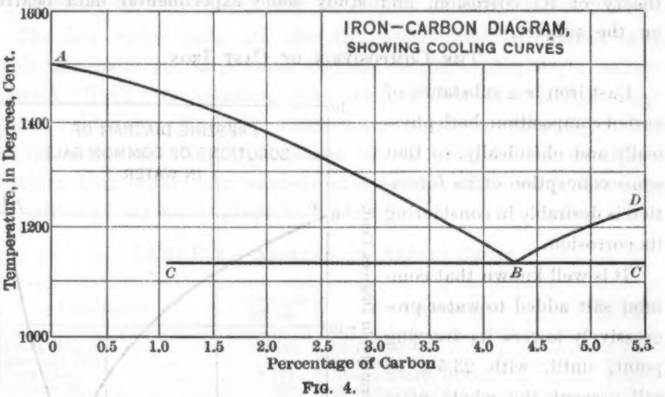


FIG. 4.

Pig iron, similarly, may be termed a frozen solution of carbon and iron. If no other impurities were present, it would contain 4.3% of carbon. Fig. 4* shows graphically how the addition of carbon lowers the solidifying point to *B*, or the eutectic, and then raises it to *D* as additional percentages are present, the line, *C-C*, indicating the constant solidifying point of the mother liquor at 1 135° cent. The free graphite found in cast iron, therefore, is due to the same action as the solid plates of salt frozen out of the water, with this important difference, that whereas ice does not retain any salt in solid solution, the solidified iron retains much carbon.

It has been known for a long time that in pig iron which has been heated considerably above its fusing point, the carbon tends

* "Cast Iron in the Light of Recent Research," Hatfield.

to separate in the graphitic form, whereas the same metal, heated to a lower degree, on cooling, would retain more carbon in the combined condition. Much study has been devoted to this obscure subject, and a series of modifications of iron and iron-carbon, all due to different conditions of heating and cooling, have been investigated, but it would be out of place to consider the subject in this paper.

To summarize briefly: the metal collecting in the bottom of a blast furnace is a saturated solution of carbon in iron, and as it cools to the solidifying point, carbide is thrown out of solution. This splits up into iron and carbon. Iron solidifies, the mother liquor splits up, and the same process is repeated. It is analogous to the behavior of the salt solution.

The carbon in a casting, amounting ordinarily to between 3 and 4%, may all be in the combined state, forming the intensely hard carbide of iron, Fe_3C, with a characteristic white fracture, or it may be in the form of free graphite, the numerous plates of which cut up and weaken the matrix of ferrite, the whole having the gray fracture found in gray iron.

Each added impurity (and many are invariably present) increases the complexity of the casting, both chemically and physically, and sets up innumerable nodes for electrolytic differences of potential.

Corrosion of Cast Iron: Explanatory Theories.

The rusting of iron is by no means the simple process it was once thought to be. How heterogeneous a casting may be, both in point of composition and in physical structure, has been shown. This adds materially to the difficulty, and explains the often apparently discordant results obtained by careful investigators.

Five separate theories have been advanced to account for the rusting of iron, but, at present, three of these have been practically eliminated, leaving only two over which the fight still wages: the acid theory and the electrolytic theory. The one undisputed fact is that iron cannot rust in air unless water is present, nor can it rust in pure water unless oxygen is present.

Mr. J. Newton Friend* has recently collated much scattered material, and has placed virtually all the researches hitherto made on this subject in the hands of the investigator. To his work, the writer is greatly indebted.

* "The Corrosion of Iron and Steel."

The Acid Theory.—This theory regards corrosion as resulting from an acid, usually carbonic, which unites with the iron to form ferrous carbonate ($FeCO_3$) or the soluble ferrous hydrogen carbonate ($FeH_2[CO_3]_2$), and the liberated hydrogen combines with any dissolved oxygen in the water to yield water. The oxygen of the air next converts the soluble iron salt into hydrated oxide, or rust, liberating the carbon dioxide (CO_2) which is now free to attack a fresh portion of iron.* With a supply of water and oxygen, a small portion of carbon dioxide could thus rust an indefinite quantity of iron.

The Electrolytic Theory.—This theory holds that the presence of an acid is not necessary to cause rusting, but regards the action as electrochemical.†

To state briefly a few of the fundamental principles, consider the case of two platinum plates immersed in a solution of common salt ($NaCl$), one plate connected to the positive and the other to the negative pole of a battery. Hydrogen will be immediately given off at the plate connected with the negative pole, and oxygen at the positive pole. If a few drops of a solution of litmus are added, the liquid around the positive plate becomes red, indicating the presence of an acid, and that around the negative pole turns blue, showing that an alkali is present. The appearance of the gases and of the red and blue color is noted at once when the battery wires are connected to the plates, even if the latter are separated by quite a quantity of solution, giving strong support to the supposition that matter travels with the current, part toward the positive and part toward the negative pole. Farraday named these moving portions of the solution "ions." A solution or a fused salt which conducts the electric current is termed an electrolyte.

If a steel knife-blade is immersed in a solution of copper sulphate ($CuSO_4$), iron dissolves and copper is deposited, the electric charge being transferred from the ions of copper to those of iron. Electrochemically, hydrogen is a metal, and is classed with copper in relation to iron. If, therefore, iron is placed in a solution containing hydrogen ions, a similar reaction will take place, iron being dissolved and hydrogen passing from the electrically charged or ionic state to the atomic

* Crum Brown, *Journal*, Iron and Steel Inst., 1888, No. II, p. 129.

† Whitney, *Journal*, Am. Chem. Soc., 1903, Vol. XXV, p. 394; Cushman, "The Corrosion of Iron," Public Roads Bulletin No. 30, Washington, 1907.

or gaseous condition,* and being deposited on the iron blade. Ferrous salts, as well as ferrous hydroxide, are rapidly oxidized to the ferric state, if air or oxygen is present.

It is a matter of common observation that, when iron rusts, it does not do so uniformly, but in patches and spots. Allerton S. Cushman, Assoc. Am. Soc. C. E., noticed that if this rusting took place in water or in a neutral solution of an electrolyte, such as salt, to which a little phenol-phthalein had been added, a pink color developed, and this was confined to certain nodes or spots. This pink color is a proof of the presence of hydroxyl ions, and thus indicates the negative poles. Professor Walker† suggested adding a trace of potassium ferricyanide to the solution, in order to show the positive poles. If iron goes into solution, ferrous ions must appear, which, with the ferricyanide, form Turnbull's blue. Walker further suggested stiffening the reagent with gelatine and agar-agar to preserve the effect produced. This reagent, which indicates at once the positive and negative poles, with the ferrous ions shown by blue at the positive and the hydroxyl ions by pink at the negative poles, was named ferroxyl by Mr. Cushman.

Later, the writer will show the behavior of some of the corroded water pipes when ferroxyl is applied.

Whatever may be the abstract facts concerning the electrolytic and acid theories, if absolutely pure iron, pure oxygen, and pure water are used, both reactions take place under actual conditions of corrosion.

Natural waters are usually charged with dissolved oxygen and carbon dioxide from the air, thus readily permitting the action of carbonic acid on the iron. Even the purest commercial wrought irons are not homogeneous, either physically or chemically, so that electrolytic action also comes into play.

Conditions Contributing to Corrosion.

Hale,‡ in studying experimentally the corrosion of iron pipes by water flowing through them, concluded that practically carbonic acid, coupled with dissolved oxygen, is a most potent factor in corrosion. The iron is dissolved as bicarbonate with the release of hydrogen:

$$Fe + 2H_2CO_3 = FeH_2(CO_3)_2 + H_2.$$

* Whitney, *Journal*, Am. Chem. Soc., 1903, Vol. XXV, p. 394; Cushman, "The Corrosion of Iron," Public Roads Bulletin No. 30, Washington, 1907.

† *Journal*, Am. Chem. Soc., 1907, Vol. XXIX, p. 1257.

‡ *Journal*, Am. Public Health Assoc., Vol. III, p. 1337.

About 20% of this hydrogen re-united with the dissolved oxygen in the water. The soluble iron bicarbonate was oxidized also by the dissolved oxygen to insoluble red oxide, setting free the carbonic acid:

$$4FeH_2(CO_3)_2 + O_2 + 4H_2O = 2Fe_2O_3 + 8H_2CO_3.$$

This freed carbonic acid again dissolves more iron and is again set free, until all the oxygen is exhausted. As a result of the catalytic action of the carbonic acid and water, a very large quantity of iron is corroded for each part per million of dissolved oxygen. Theoretically, 10 parts per million of dissolved oxygen produce 126 parts per million of Fe_3O_4, expressed as Fe; whereas, if the hydrogen were oxidized and not set free, the quantity oxidized by 10 parts of oxygen would be only 31 parts per million of iron. By experiment, Hale found that from 75 to 80% of the theoretical quantity of iron was actually oxidized.

Although, theoretically, the oxide is the final stage, the differing colors of red, green, brown, and black indicate that the iron exists in hydrated form, except when formed in the hot way.

Hale also found that the rate of corrosion increased proportionately to the increase of free carbonic acid. Analyses for soluble iron and dissolved oxygen proved that this increase was due to the fact that the carbonic acid dissolved the iron more rapidly, and more iron being in solution, oxidation is hastened.

The aqueous solutions of most acids dissolve iron. For example, if hydrochloric acid is used, ferrous chloride and hydrogen are the resultant products, the former oxidizing into ferric chloride in the presence of air. If only a trace of acid is present, the iron will be deposited from the solution as hydrated ferric oxide or rust.

Cast iron subjected to the slow action of dilute acids, as, for example, acid water from mines, retains unchanged the external shape of the casting, but the composition of the metal is altered, iron being removed in solution.

Very dilute solutions of alkaline hydroxides, such as sodium hydroxide (NaOH), or caustic soda, and calcium hydroxide ($Ca[OH]_2$), or slaked lime, rapidly absorb carbon dioxide, forming carbonates which in such dilute solution do not prevent the iron from rusting. On the contrary, by hastening the absorption of oxygen by the iron, they increase corrosion.

If a plate of iron is cleaned, weighed, and placed in a beaker with either water or a solution of a salt, and after remaining in the solu-

tion for some fixed period, is removed, cleaned, dried, and weighed, the loss in weight measures the quantity of corrosion.

Taking 100 as the rate of corrosion of pure iron in water, Fig. 5 shows the rates in varying strengths of salt (NaCl) and sodium sulphate ($Na_2SO_4 10H_2O$), according to the experiments of Friend and Brown.* The surprising fact is that, whereas dilute solutions of these salts increased the corrosive action of water, stronger solutions exerted less corrosive action on pure iron than water alone. This bears out the experiments of Adie, made some 69 years ago.† He took weighed pieces of wrought and cast iron; immersed some in pure water and some in saturated solutions of salt, and after the lapse of several days, re-weighed them, with the results shown in Table 3.

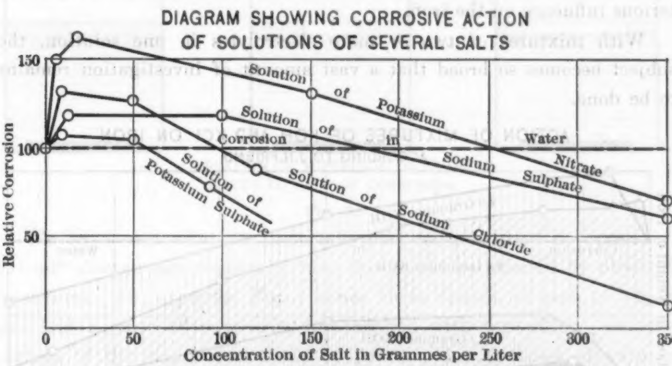


FIG. 5.

TABLE 3.

Conditions of the experiments.	Loss in fresh water, in grains.	Loss in salt water, in grains.
Twenty pieces of wrought iron, weighing 374 grains, exposed for 80 days..................	1.9	0.1
Three rods of cast iron, weighing 787 grains, exposed for 62 days........................	1.6	0.4

Both kinds of iron were corroded much more in the fresh water than in the saturated solution of salt water. Adie attributed this to the decreased solubility of oxygen in salt solutions. Friend proved experimentally that, in the absence of air, solutions of sulphates and chlorides of sodium and potassium have no action on pure iron.

* "Corrosion of Iron and Steel," by J. Newton Friend, 1911.

† *Minutes of Proceedings*, Inst. C. E., 1845, Vol. IV, p. 323.

Rain-water, particularly in thunder-storms, contains nitric acid or nitrates, and polluted waters also may contain nitrates and nitrites in solution. The curves on Fig. 5, showing the corrosive action of potassium nitrate and of potassium sulphate, also from Friend and Brown, are interesting, the more so as it is very dilute solutions of these substances which are ordinarily encountered. As these curves show, their corrosive effect is then at a maximum.

Magnesium and ammonium chlorides are found to corrode iron, even in the absence of air. This gives a clue to the particularly destructive effect of sea water (which contains magnesium chloride) on cast iron, even at great depths below the surface, where fresh water would not contain enough dissolved oxygen to exert any deleterious influence on the iron.

With mixtures of two or more electrolytes in one solution, the subject becomes so broad that a vast amount of investigation remains to be done.

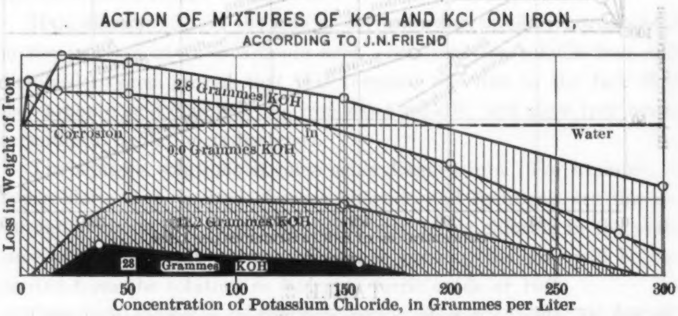
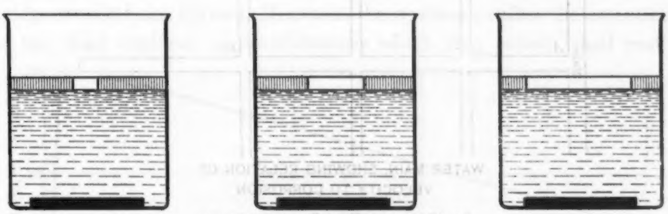


FIG. 6.

It is very important, however, to have some idea of what occurs under such circumstances. Friend* placed 36 pieces of pure iron foil in a series of hard glass beakers, the first containing water, and the others containing water in which varying quantities of potassium chloride and potassium hydroxide (caustic potash) were dissolved. These were allowed to remain 28 days, after which the iron plates were removed, cleaned, and weighed. The results are shown graphically by Fig. 6. From this it will be seen that pure water corrodes iron much more rapidly than water containing caustic potash; that

* *Journal*, Iron and Steel Inst., 1911.

the corrosive action is greatly increased by the presence of potassium chloride; that increasing quantities of chloride first increase and then gradually diminish the intensity of corrosive action; and that increasing quantities of caustic potash, after a temporary increase in dilute solution, diminish corrosive action, finally inhibiting it altogether. It is this action of alkalis which explains the protection of steel and iron in concrete, the alkalis in the cement exerting a protective action, in spite of the simultaneous presence in the cement of corrosive electrolytes such as sulphates of magnesium and alkaline earths.



RELATION OF SUPERFICIAL AREA OF WATER TO RATE OF CORROSION

FIG. 7.

We have seen what an important part oxygen plays in corrosion. Friend* showed experimentally how this might be affected by physical conditions. For example, Fig. 7 shows three vessels, in each of which was laid a piece of pure iron foil covered with tap water. On the surface of the water were disks of paraffin, having holes of various sizes cut in them, exposing progressively smaller areas of water surface to absorb oxygen from the air. After 5 days, the pieces of iron foil were weighed, and the relative results are given in Table 4.

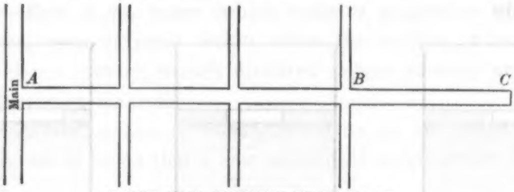
TABLE 4.—RELATION OF SUPERFICIAL AREA OF WATER TO RATE OF CORROSION.

Superficial area of water, in square centimeters.	Relative corrosion.
56.8	100
12.6	58
0.8	14

The rate of motion of water also has a bearing on the rapidity of corrosion. In certain circumstances, it causes greater aeration,

* "The Corrosion of Iron and Steel." p. 75.

and it also brings a constantly renewed supply of air to the rusting metal. Mr. M. B. Jamieson* gives an interesting example of this, illustrated in Fig. 8, in which a main, running from a larger main at *A* to a dead end at *C*, had a number of side branches taken off it. There was, therefore, a continually diminishing quantity of water and a diminished rate of flow, the greater the distance from *A*. After 45 years of use, the pipe was found almost choked with rust at *A*, but with less and less of it as the point *C* was approached. The pipe was practically clean from *B* to *C*.



WATER MAIN, SHOWING RELATION OF VELOCITY TO CORROSION

FIG. 8.

An interesting series of investigations showing this relation was undertaken by Heyn and Bauer,† and is quoted by Friend. Two weighed plates of cast iron were hung from a bent glass rod in a beaker and covered with water. A stream of running water was admitted by a glass tube and the excess was allowed to run away over the sides of the beaker, the operation being continued for a period of 3 weeks. Several beakers were used, the rate of flow being different in each, all other conditions being the same. At the end of the experiment, the plates were cleaned and weighed, the loss showing the corrosion. Fig. 9 shows the results of these experiments. It will be seen that a gentle flow increases the corrosion to more than eight times that in still water, but that it gradually diminishes as the rate of flow increases.

It is a noticeable fact that in most cases of bad pipe corrosion that have come to the writer's notice, not only was there salt water, aeration, and more or less motion in the water, but the pipes were laid, as a rule, in muck with rank swamp vegetation and life in close contact with the metal. The action of living matter, therefore, becomes

* *Minutes of Proceedings*. Inst. C. E., 1881, Vol. LXV, p. 323.

† *Mitteilungen* aus dem Königlichen Materialprüfungsamt. Berlin, 1910, Vol. XXVIII, p. 93 *et seq*.

a subject of interest, and will be found to merit careful consideration in many instances in which it has not hitherto been given a thought.

The direct action of any organism is doubtful, although it is known that organisms do occur in the tubercles forming on the inside of water mains. Dr. Brown* states that "Organisms are found in some incrustations and may modify their composition; on the other hand, incrustations are often free from every trace of life." Mr. Richard Gaines† found it probable that certain sulphur bacteria were responsible in part at least for rust tubercles and pitting in conduits. He also cites a case in which, during repairs to the foundations of a bridge across Lake Hauser, Montana, the workman called the attention of the chief engineer to protuberances which they called "shell rust"

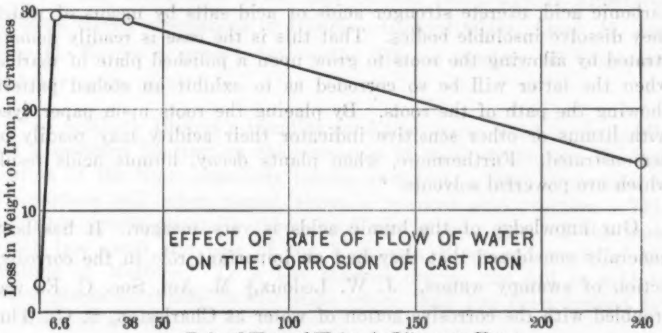


FIG. 9.

and which were found scattered over the surface of submerged steelwork. Schorler identified in this rust *Gallionella ferruginea* in large quantities. Apparently, the iron is dissolved by some acid secretion from these organisms, and, in this instance, the structural damage to this bridge which had only been in service for one year, had been enormous. It is even necessary to consider the earthworm. It has been estimated that, on an average, some 25 000 worms exist in every acre of soil. Darwin‡ concludes from his observations that each worm annually ejects 20 oz. of soil as castings, which means that 15 tons of soil in every acre of land pass through the worms each year. These castings are acid when fresh, and contain also small quantities of

* *Minutes of Proceedings*, Inst. C. E., Vol. CLVI.
† *Chemical News*, 1910, Vol. CI, p. 205.
‡ "Vegetable Mould and Earthworms."

ammonia. With 15 tons of acidified soil per acre per annum, with a small admixture of ammonium salts, there may well be a corrosive action on any embedded iron.

Darwin placed some worms in a pot of sand which, owing to the presence of ferric oxide, was red in color. The sand which passed through the worms and was found in their burrows, was bleached, and on microscopic examination, it was found that the ferric oxide was completely dissolved. Acetic acid scarcely produced any effect on this sand, yet the acid secretions of the worms decolorized it effectually.

Of still greater importance, probably, is the action of plants:

"Young roots, and especially root hairs, in addition to exhaling carbonic acid, excrete stronger acids or acid salts by means of which they dissolve insoluble bodies. That this is the case is readily demonstrated by allowing the roots to grow upon a polished plate of marble, when the latter will be so corroded as to exhibit an etched pattern showing the path of the roots. By placing the roots upon paper dyed with litmus or other sensitive indicator their acidity may readily be demonstrated. Furthermore, when plants decay, humus acids result which are powerful solvents."*

Our knowledge of the humic acids is very meager. It has been generally considered that they had an important rôle in the corrosive action of swampy waters. J. W. Ledoux,† M. Am. Soc. C. E., was troubled with the corrosive action of water at Charleston, S. C. This water was obtained from a reservoir covering 2 300 acres and submerging a large quantity of peaty matter, much of it 15 ft. deep, a large part of the water-shed of some 49 sq. miles being low and swampy. He made a series of tests by boiling weighed pieces of piano wire in 500 cu. cm. of the water. This was done with raw water, water previously boiled to free it from oxygen and carbonic acid, and water to which various alkalis had been added. The loss in weight of the wire determined the relative corrosion. He was

"unable to determine whether ionization is the cause of the trouble, but rather believes in the theory that the raw water contains organic compounds which are corrosive in effect and with the elimination of organic matter as the result of the use of sulphate of alumina and filtration, these effects largely disappear."

* Friend, "Corrosion of Iron and Steel," p. 105.
† *Engineering Record*, Vol. LX, p. 701.

The indirect action of humus may be extensive under certain conditions. Jodidi* refers to the fact that the amount and nature of humus formed depends on the organic materials which undergo humification, on temperature, moisture, aeration of soil, presence of salts, acids, etc., and the character and quality of microbes present. The comparatively easily soluble products are leached out, and the less soluble ones accumulate in the humus. The humus is further oxidized by the action of air; the carbon to carbon dioxide; the hydrogen to water; the nitrogen to nitric acid; and the sulphur to sulphuric acid; and putrefaction leads to a number of secondary products, such as amino acids and acid amides.

How important these acids may become practically is illustrated by some recent experiences in Illinois. Clifford Older, Assoc. M. Am. Soc. C. E., in a paper† before the Illinois Society of Engineers and Surveyors, says:

"In southern Illinois a peculiar condition exists which undoubtedly has a material bearing on the corrosion of steel structures. In that portion of the State popularly known as Egypt, a large proportion of the surface soil, when tested, shows a marked acid reaction. This condition exists generally throughout the greater part of the state which lies south and east of the Kaskaskia River and south of the latitude of Mattoon. To a less marked degree, the soil is acid in many places elsewhere in the state where land has been farmed for 50 years or more.

"This acid condition does not exist for a depth of more than a few inches, and steel buried at a greater depth does not seem to exhibit any unusual tendency to corrode."

He notes the complete destruction of the webs of I-beams and channels having a thickness of ¼ in. in from 10 to 12 years.

Dr. Adeney ascribes the corrosive action of the Vartry water supply of Dublin to the presence of minute quantities of peaty matter, which ferment slowly, with the formation of carbonic acid, as well as small quantities of nitric acid.

The destructive effect of carbonic acid has already been referred to; in swampy waters, for the reasons mentioned, it is high, and, in many well waters, it is very high. It is, moreover, very soluble in

* "The Chemistry of Humus." *Journal* of the Franklin Inst., Vol. 176, p. 565.

† *Engineering News*, Vol. LXXI, p. 354.

water, so that it is in available form to attack any iron exposed to its influence.

It can be readily understood, therefore, that, by their various secretions, plants and animals are important accessories in the corrosion of iron.

Conditions Inhibiting Corrosion.

In reviewing the conditions contributing to corrosion, it was shown that whereas dilute solutions of alkalis did not retard corrosion, strong solutions neutralize the acids and preserve the iron for an indefinite period. Mr. Allerton S. Cushman found that 5% of lime applied to boggy, sour land exerted a pronounced retarding action on the corrosion of ironwork embedded in it.* The same authority further says:

"There should be many cases where the property of alkalis to inhibit corrosion could be made of more practical use than has been done. Whenever iron posts or standards are set directly in the ground, the liberal use of slaked lime should be beneficial."

The effect of concrete in protecting iron and steel was referred to when discussing the action of two or more electrolytes in one solution.

Cast iron is obtained by pouring the molten metal into sand moulds. The outer skin, therefore, is silicious. It has long been known that silicon tends to protect iron from corrosion, and Jouve† has shown that a 20% alloy of iron and silicon is not attacked by acids, but is virtually uncorrodible. Wallace‡ refers to having seen natives in India forge iron on a stone anvil, and says this iron, which was charcoal iron, does not rust on exposure to the weather, but takes on a fine brown patina. The question was raised whether the stone anvil did not siliconize the skin of the iron. In the experiments by Messrs. McCollum and Logan§, it was found that, under a given length of exposure in soil, if the corrosion of machined cast iron be considered as 100, that of cast iron still retaining the original surface as cast would be only 93. Krohnke‖ also notes the fact that the surface formed when pipes are cast, enables them to resist attack for a long time.

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* *Journal*, Iron and Steel Inst., 1909, Vol. LXXIX, p. 33.

† Friend, "Corrosion of Iron and Steel," p. 229; Jouve, *Journal*, Iron and Steel Inst., 1908, Vol. LXXVIII, p. 310.

‡ *Journal*, Iron and Steel Inst., 1908, Vol. LXXVI, p. 84.

§ "Electrolytic Corrosion of Iron in Soils," p. 31.

‖ *Gesundheits-Ingenieur*, Vol. XXXIII, p. 393.

FIG. 10.—FOUNTAIN OF NEPTUNE AT VERSAILLES.

FIG. 11.—BEHAVIOR OF TIN (A) AND ZINC (B) IN RETARDING CORROSION OF IRON.

Thwaite prepared a table (reproduced as Table 5) from various published data, showing well the protective effect of this skin.

TABLE 5.—RELATIVE CORROSION OF CAST IRON WITH AND WITHOUT "SKIN".

Metal.	Foul sea water.	Clear sea water.	Foul river water.	Pure air and clear river water.	City air or sea air.
Cast iron................	3.500	3.386	2.084	0.604	2.637
Cast iron (skin removed by planing)...........	12.275	4.738	3.884	0.584	4.763

Silicon is an undesirable alloy for iron in many ways, so that a surface treatment, such as naturally occurs in casting, is probably the only method of making use of its rust-inhibiting properties.

Chromium, nickel, and copper, when present in small quantities, contribute greatly to the resistance of iron to corrosion. A steel containing about 1% of chromium was found by Hadfield to corrode about half as much as ordinary mild steel. Similarly, 3% of nickel added to steel reduced its relative corrosion in salt water as compared with open-hearth steel, about 50 per cent.*

Phosphorus in iron also retards corrosion, differing in this respect from sulphur, which, by forming sulphuric acid, powerfully attacks the metal.

The coating of iron with zinc and tin is also used to inhibit corrosion. Zinc is electro-positive to iron, and if the two metals are in contact, and are immersed in a corroding solution, the zinc will pass into solution and the iron will remain unaffected. Galvanizing, therefore, is not merely a mechanical shield to the iron, but protects the metal for some distance around it. It is well to emphasize this fact, as it is a point frequently misunderstood. In a pocket manual of tables and information for engineers, bearing the date of 1911, this statement occurs: "With both zinc and iron exposed and in contact with water, corrosion proceeds more rapidly than it would if the zinc were not present." This statement is absolutely misleading and erroneous.

Tin, on the contrary, is not electro-positive to iron. Being itself practically unaffected by corrosion, it protects the iron mechanically. The tin coating, as commercially applied, is full of minute pin holes.

* Friend, "Corrosion of Iron and Steel," pp. 217-225.

It does not inhibit corrosion of the iron at these points, thereby differing from zinc. Fig. 11 shows well the dissimilarity between the two metals in this respect. A plate of polished sheet iron was placed in a dilute solution of salt and water. At *A* was laid a lump of granulated tin and at *B*, a lump of granulated zinc. In the photograph may be seen the area of polished iron free from rust around the lump of zinc, whereas the tin had absolutely no effect in protecting the iron in its vicinity.

A Study of Typical Cases of Deterioration of Cast-Iron Pipe.

With these various facts in mind, an examination of certain typical cases of pipe failure may give information of value to the engineer who is called on to design works involving pipes subjected to deleterious influences.

It has already been noted that most, if not all, pipe failures occur from the influence of salt or saline waters.

A 10-in. main runs southward from Elizabeth, N. J., through the meadows to Morse's Creek. At this point, it is continued by a 6-in. line, also running southward. The general elevation of the meadow, as previously mentioned, is about 18 in. above mean high tide, and the bottom of the pipe is approximately at the level of mean high tide. There is no trolley near. Curiously enough, the 10-in. pipe, which was laid in 1893, has shown no appreciable deterioration except at a point opposite the works of the Standard Oil Company, where escaping acid waters caused corrosion. The 6-in. line, although not laid until 1902, has broken down completely. Both sizes were made by the Warren Foundry Company, and there is no apparent reason for the greater resisting power of the 10-in. line.

The salt marsh is intersected by creeks and ditches, and the surface is largely covered with fox grass. There are patches too wet for the fox grass, and these are covered with what is known as quill sage, a coarse, rank, swamp growth. It is in these patches that the chief damage to the pipe occurs. Fig. 12 illustrates the character of these meadows. The coarseness of the quill sage distinguishes the patches covered by it from those on which the finer fox grass is found.

Fig. 13 shows a photograph of a ring of the 6-in. pipe, and Fig. 14 shows a section of this pipe. In the ring, two places deeply corroded will be noticed at "B", and yet the exterior is smooth, to casual ex-

FIG. 12.—SALT MEADOW, ELIZABETH, N. J., SHOWING COARSE PATCHES OF QUILL SAGE AMONG THE FINE FOX GRASS.

FIG. 13.—RING FROM 6-INCH CAST-IRON MAIN, ELIZABETH, N. J., SHOWING CORROSION.

amination sound, and is neither swelled nor pitted, but retains its original sharp outline. At "A" the same thing is noticed, except that a slight crack occurs between the sound and corroded parts in one place. Just below "A", the corrosion extends nearly through the pipe. This condition exists in the section shown in Fig. 14, where the pipe has been completely penetrated. The interior is smooth and not tuberculated.

The corroded portion has the general appearance and consistency of graphite, and cuts like that substance. When first removed from the swamp, it is much softer, more like soft putty. When it is pulverized for analysis it is brown, and makes a brown streak. It is possible to write on paper with a fragment of the pipe, the marks resembling those made by a brown crayon.

If some filings of the unchanged metal are sifted on a card beneath which is placed a horse-shoe magnet, they arrange themselves in magnetic curves, as shown to the left of Fig. 15. Filings from the corroded part, on the contrary, if also sifted on a card with the magnet beneath, fall as an inert mass, which is shown on the right of Fig. 15. From this it will be seen that, when carried to completion, the iron is completely changed from the metallic state, and that which is not leached out as chloride, remains as hydroxide.

One piece of the corroded portion contained only 42.2% of iron. Another portion had the composition given in Table 6, according to the analysis by Mr. Edward J. Pugh.

TABLE 6.—Corroded Portion of Pipe from Elizabeth.

Iron	50.62	per cent.
Silica	0.31	"
Chlorine	0.72	"
Manganese	0.46	"
Silicon	22.32	"
Combined carbon	0.63	"
Graphitic carbon	5.51	"

At a part not very deeply corroded, the specific gravity of a complete cross-section of the pipe was 5.18. The specific gravity of a piece from the corroded part of the pipe was only 2.2. The specific gravity of cast iron, as given by Trautwine, varies from 6.9 to 7.4, and is assumed to be 7.15.

Three microscopic views of a polished section of the uncorroded pipe are shown in Figs. 16, 17, and 18. Fig. 17 is magnified 50 diam-

eters, Fig. 18 40 diameters, and Fig. 16, approximately, 150 diameters. The thin plates of free carbon referred to in describing the composition of cast iron can readily be seen, interspersed with the ferrite. Fig. 19 shows a micrograph of a portion of the corrosion magnified 40 diameters. Owing to the softness of the material, it is impossible to obtain a high polish on these specimens, so that the view is somewhat obscured by the shadows, but, in Fig. 20, where the carbon is indicated by heavy lines, it will be seen that the graphite plates are much thicker than they are in the unchanged metal.*

Fig. 21 shows in plan and section a piece of the pipe corroded in spots. In this, as in the other examples, the sharp original outline of the metal remains even after the metallic iron has been completely removed or oxidized. A paraffin dam was placed around this piece of pipe, and ferroxyl reagent poured on. The lower part of Fig. 21 shows the result. Where there was a deep corrosion, as at the bottom of the figure, no change took place. The iron alongside this developed two red nodes, showing the presence of hydrogen ions, and a wavy cloud of blue indicates where the iron is passing into solution. A section of pipe, similar to Fig. 14, when placed in a petri dish with ferroxyl, strongly developed the red color all along the outside or deeply corroded part, and a perfect mass of blue from all the uncorroded portion. This action is much more vigorous than that undergone by either a steel wire nail, a wrought-iron nail, or a piece of cast-iron soil pipe lying in the same petri dish. The chlorine present in the corrosion (as shown by Table 6) doubtless accounts for the vigor of the reaction observed.

SECTION OF CORRODED PIPE TAKEN FROM SALT MARSH, ELIZABETH, N.J.

FIG. 14.

At Atlantic City the behavior of the pipe was similar to that at Elizabeth. The conditions also were analogous, the chief difference being that the salt meadow was only 6 in. above mean high tide, so that the pipe was subjected to more frequent alternations of water and moist air.

Two samples of soil were collected by the writer from the Atlantic City meadows, one of them ½ in. from a corroded part of the cast-iron

* See page 816, comments on analysis of corroded pipe at Syracuse.

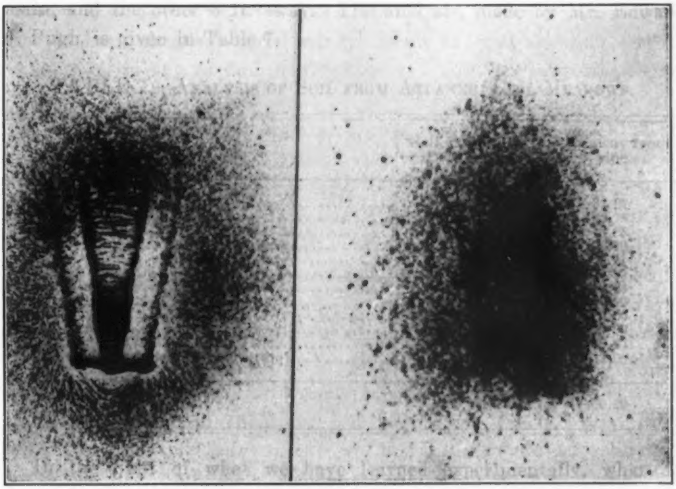


Fig. 15.—Behavior Toward the Magnet of Filings from Corroded and Uncorroded Parts of Pipe Compared.

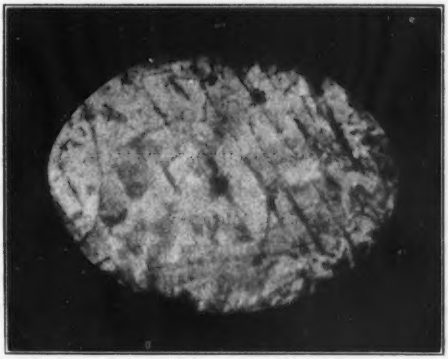


Fig. 16.—Section of Cast-Iron Pipe, Elizabeth, N. J., Magnified 150 Diameters.

main, and the other 3 ft. away. The analysis, made by Mr. Edward J. Pugh, is given in Table 7.

TABLE 7.—ANALYSIS OF SOIL FROM ATLANTIC CITY MEADOWS.

	Soil within ½ in. of pipe.	Soil away from pipe.
SiO_2 (insol. in HCl)	18.68	32.27
Al_2O_3, CaO, etc.	10.57	22.78
SiO_2 (sol. in HCl)	0.28	0.28
Al_2O_3 " " "	1.91	1.19
Fe_2O_3 " " "	46.57	9.11
NaCl	1.98	0.95
CaO (sol. in HCl)	0.99	0.83
MgO " "	0.00	0.00
Sulphates	0.00	0.00
Organic acids	0.00	0.00
H_2O (at 105° cent.)	6.28	28.92
Volatile matter besides H_2O (ignition)	13.27	3.96
	100.43	100.29

In the light of what we have learned experimentally, what do these observations on the corroded pipe signify, and how shall they be interpreted?

It has been shown (page 823) that aqueous solutions of most acids dissolve iron. If hydrochloric acid is used, ferrous chloride and hydrogen are the resultant products, the former oxidizing into ferric chloride in the presence of air. It has also been shown that, if only a trace of acid is present, the iron will be deposited from the solution as hydrated ferric oxide. It is probable* that in salt water the reaction may be represented by the equation:

$$Fe + 2NaCl + 2H_2O = FeCl_2 + 2NaOH + H_2.$$

In the presence of air or of dissolved oxygen in the water, this free hydrogen oxidizes to water, the iron in solution yields ferric hydroxide, and the process continues. The chemical analysis of the portion practically free from metallic iron contains as low as 42.2% of that element, a small part of which is in the form of chloride. Clearly, part of the iron has been removed in solution, and part remains as the hydroxide, the carbon present in the combined form being left behind, and, as shown in the microscopic sections, has thickened the plates of free carbon originally in the free iron.

* Friend. "Corrosion of Iron and Steel," p. 147.

The oxides of iron are electro-negative to the metal, so that electrolysis plays its part, as shown by the ferroxyl reagent, and the extraordinary vigor of the reaction is illustrated by the great mass of blue produced.

The chlorides of sodium and potassium, as heretofore mentioned, cannot corrode iron in the absence of air. Magnesium chloride, which is present in sea water in small quantities, is able to do so, but to get the full corrosive effect of sea water, plenty of air is essential.

It has been shown (Fig. 7) that the greater the surface of liquid exposed to the air, the more the corrosion is intensified.

In the salt marshes in question, the tide rises and falls, passing through a multitude of creeks and ditches all over the meadows. At Elizabeth, it ordinarily reaches about to the bottom of the pipe; at Atlantic City, it covers it. As shown in Fig. 22, this affords an ideal condition for corrosion. The liquid at high tide touches or surrounds the pipe, as in *A*; at low tide, it falls, air rushes in to take its place, and a moist, saline, oxygenated, spongy mass of earth and roots surrounds the pipe. Carbonic acid probably contributes to the action. The Atlantic City meadow soil was free from organic acids. In the sample of soil taken ½ in. from the pipe, there was 46.57% of Fe_2O_3 and even 3 ft. away, iron occurred in large quantities, there being 9.11% of Fe_2O_3. This would appear to show that it had been dissolved from the pipe as $FeCl_2$, and, after diffusing or flowing away from the pipe, as the tides rose and fell, was deposited as hydroxide.

It is further to be noted that at Elizabeth, where, on account of the elevation of the meadow, the top of the pipe was not so frequently wet as the bottom, the blow-outs generally were in the bottom of the pipe. At Perth Amboy, the top and upper sides suffered most. At Atlantic City, also, the condition of the pipe was worse at the top than at the bottom. This was to be anticipated on account of the relative elevations of the pipe and high tide. At Elizabeth, the high tide reached the bottom of the main; consequently, the top was not subjected to the saline influences so frequently. At Atlantic City, the pipes, ordinarily, were covered at high tide, and, being laid farther below the surface, air was more plentiful at the top, which, therefore, in presence of an ample quantity of salt water and with a more abundant supply of oxygen than the bottom, suffered most.

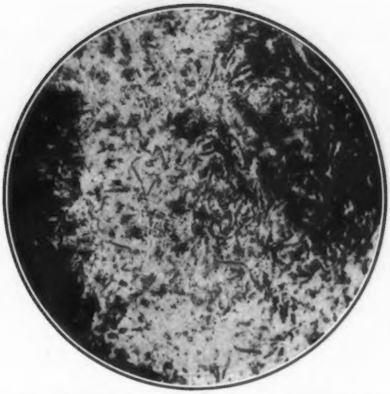


FIG. 17.—SECTION OF CAST-IRON PIPE, ELIZABETH, N. J., MAGNIFIED 50 DIAMETERS.

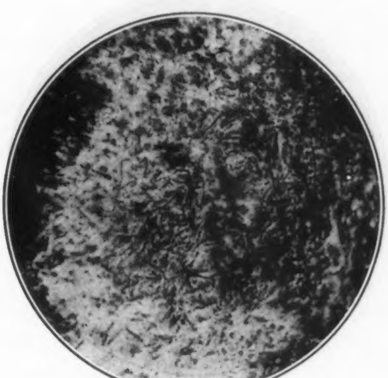


FIG. 18.—SECTION OF CAST-IRON PIPE FROM ELIZABETH, N. J., MAGNIFIED 40 DIAMETERS.

Fig. 19.—Section of Corroded Portion of Pipe from Elizabeth, N. J., Magnified 40 Diameters.

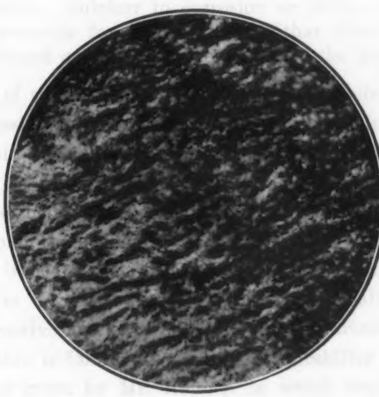


Fig. 20.—Section of Corroded Portion of Pipe from Elizabeth, N. J., Magnified 40 Diameters, Showing Carbon Plates.

At Syracuse, the portions constantly submerged did not give out. Probably, too, the concentrated saline content in these brine-soaked lands produced an inhibiting effect, as shown in Fig. 3. The damage took place above the permanent water table, where both air and salt water were enabled to reach the pipe.

Analyses of the corroded and uncorroded portions of the Syracuse pipe were made for the Water Department by Dr. Ernest N. Pattee, of Syracuse University, and are given in Table 8.

TABLE 8.—ANALYSES OF CORRODED PIPE FROM SYRACUSE, N. Y.

	Non-corroded portion.	Corroded portion.
Silicon	0.15 per cent.	1.49 per cent.
Sulphur	0.11 " "	0.20 " "
Phosphorus	1.26 " "	1.39 " "
Manganese	0.30 " "	0.38 " "
Total carbon	3.65 " "	4.21 " "
Graphitic carbon	2.85 " "	3.84 " "

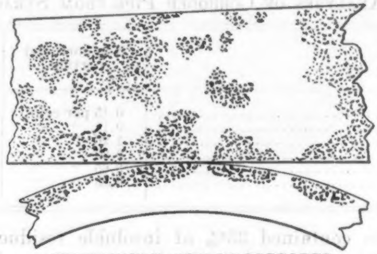
"The corrosion contained 33% of insoluble residue consisting of about 10% graphitic carbon and 22% of silicon (Si) and silica (SiO_2). Total carbon in the corrosion = 11.2%, consisting almost entirely of graphitic carbon. Sulphur in corrosion = 3.0%. Apparently, the process which accounts for the corrosion either dissolves the greater part of the combined carbon or converts it into the graphitic variety."

An analysis of the soil surrounding this pipe, made by Mr. Edward J. Pugh, disclosed that it was acid to phenol-phthalein, and contained 3.42% of organic acids. This is interesting, from its analogy to the soil conditions in Illinois, cited by Mr. Older, where steel bridges were damaged so badly by the acid soil.

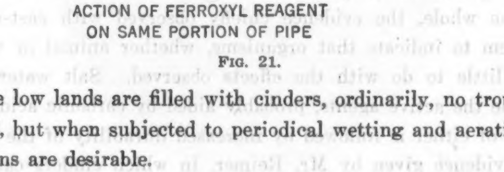
On the whole, the evidence chiefly observed with cast-iron pipe would seem to indicate that organisms, whether animal or vegetable, had but little to do with the effects observed. Salt water and air seem to be the active agents, probably aided by carbonic acid, and the exclusion of either is followed by increased durability of the pipe.

The evidence given by Mr. Reimer, in which cinders caused pipe corrosion, the similar experience at Syracuse, and the German case resulting presumably from local electrolysis due to the presence of gypsum crystals, indicate the necessity for caution where pipe is likely to be subjected to such influences. Some years ago, a tunnel leading from the Susquehanna River to the power-house of the Cen-

tral Pennsylvania Traction Company, at Harrisburg, was constructed under the supervision of Mason D. Pratt, M. Am. Soc. C. E. This tunnel passed through the property of the Central Iron Works, on parts of which there were large slag dumps. The water flowing into the tunnel was so sulphurous from the sulphur in the slag, that it greatly impeded the progress of the work and was most unwholesome for the workmen. Cast-iron pipe in such a location would undoubtedly undergo rapid deterioration.



SECTION AND VIEW OF CORRODED PIPE FROM ELIZABETH, N.J.



ACTION OF FERROXYL REAGENT ON SAME PORTION OF PIPE

FIG. 21.

Where low lands are filled with cinders, ordinarily, no trouble need be feared, but when subjected to periodical wetting and aeration, some precautions are desirable.

Acid mine waters are also very corrosive, and should be guarded against, when pipe is subject to their action. In the mines themselves this would generally be economically impracticable, but, in municipal works in the vicinity, where the pipe may be attacked, preventive means should be resorted to.

Prevention of External Corrosion.

A recipe to prevent rusting of iron, given in an old work called "The Laboratory", published in London, in 1790, is as follows:

"Fry a middling eel in an iron pan, and when brown and thoroughly fryed, express its oil, and put into a phial, to settle and become clear in the sun. Iron work, anointed with this oil, will never rust, although it lay in a damp place."

The actions of iron under the complex conditions obtaining in Nature are so various, and the observed results so often discordant, that one would fain fall back on some such recipe as this as a cure-all.

Bearing in mind the silicious skin of a cast-iron pipe and its protective effect, and also noting that chromium alloys of iron are resistant to acid attack, it occurs to the writer that some chromium compound such as chromite mixed with the facing sand of the moulds might produce in castings a protective chromium skin.

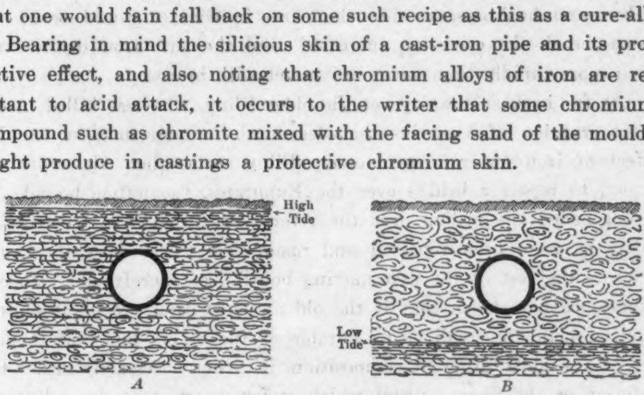


DIAGRAM SHOWING AERATION EFFECTED IN A TIDAL MARSH
A—HIGH TIDE *B*—LOW TIDE

FIG. 22.

It is a subject wherein an apparently well-reasoned argument may be completely upset. For example, as zinc in water, either fresh or salt, protects iron and is itself attacked, so if iron and copper are immersed in a liquid capable of acting on each, and are connected by a conductor, a current will pass from the copper to the iron, and the iron will be dissolved. And yet, after a series of exhaustive experiments, Mr. Richard H. Gaines* states:

"The legitimate conclusion from the foregoing experiments is that rather less than more galvanic action arises from contact between steel and the other metals of the series than from contact between steel and steel. In actual practice it has been found that if a steel nut is put on a steel bolt and subjected to corrosive conditions, in a comparatively short time the steel nut will have to be hammered or

* *Engineering Record*, Vol. LXIX, p. 104.

split off, if gotten off at all, on account of rusting together from local corrosive action; whereas a brass or bronze nut on a steel bolt under similar condition can be readily screwed off."

Mr. H. J. Force* is authority for the statement that cast iron used in pumping machinery by the Delaware, Lackawanna and Western Railroad Company, was badly corroded by acid mine waters. As bronze bushings, valves, and impeller blades were used, it was thought that the action was due to galvanic action from contact of the two metals in the acid water. On substituting cast iron for the bronze valves and bushings, so that the parts in contact would be of the same metal, the corrosion increased, and the cast iron deteriorated even more rapidly than when in contact with bronze.

As far back as the days of the elder Pliny, who was killed in the same eruption of Vesuvius that destroyed Pompeii in A. D. 79, this effect of iron on iron was noted. Pliny states that Alexander the Great, to repair a bridge over the Euphrates, "sometime bound and strengthened the bridge over the river there; the links whereof, as many as have been repaired and made new since, do gather rust, whereas the rest of the first making be all free therefrom." Modern experience has shown that if the old and new iron are of absolutely similar metal, one will be as durable as the other; but that if they are of a slightly different composition, it is not necessarily either the inferior or the newest metal which suffers most, that depending on the electrical phenomena resulting.

It is unsafe to reason too positively from results obtained with liquid electrolytes as to what will happen to iron buried in soils. Moreover, time is such an important factor that conclusions drawn from accelerated tests may be most misleading. At the beginning of this paper, reference was made to the conclusions of Messrs. McCollum and Logan.†

"The relatively high rate of self-corrosion of cast iron as compared to the other kinds of iron tested is contrary to the generally accepted idea that cast iron is more resistant to self-corrosion than wrought iron. It is not improbable that this impression in regard to the superiority of cast iron has grown out of the fact that cast iron structures are usually made relatively heavy and they also tend to corrode more uniformly than wrought iron or steel, both of which factors would tend greatly to increase the life of the former."

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* *Engineering Record*, Vol. LXIX, p. 104.

† "Electrolytic Corrosion of Iron in Soils," p. 32.

The writer cannot subscribe to this view. The ordinarily accepted idea of the superior durability of cast iron arises, in his opinion, from the fact that cast iron, generally speaking, is more durable than other commercial forms of wrought iron and steel.

One explanation of this discrepancy may be furnished by the experiments of K. Arndt,* who determined the extent of corrosion in iron and steel by the oxygen absorbed by the metal in rusting. Fig. 23 shows the result of these experiments.

The cast iron at first corrodes as rapidly as the mild steel, and more than twice as fast as the weldless tube. At the end of 43 days, however, the mild steel had undergone twice as much oxidation as the cast iron, and the weldless tube four times as much.

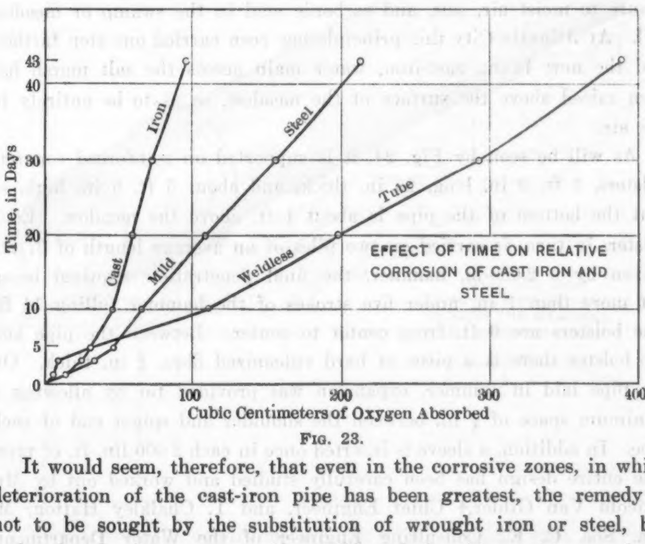

FIG. 23.

It would seem, therefore, that even in the corrosive zones, in which deterioration of the cast-iron pipe has been greatest, the remedy is not to be sought by the substitution of wrought iron or steel, but rather by the improvement, if possible, of the skin resistance of the metal. Where, for any reason, such as with flanged pipe, bolts and nuts are necessary, bronze need not be feared on account of its supposed galvanic effect on the iron.

A second method of protecting the pipe, based on the well-known rust-inhibiting properties of the chromates and certain other substances, whereby the iron is rendered "passive", does not seem prac-

* *Chemiker Zeitung*, 1910, Vol. XXXIV, p. 425.

ticable, in that a sufficient quantity of such material to protect the iron indefinitely could scarcely be added to the soil surrounding the buried pipe at a reasonable cost. Therefore, this interesting phase of the subject has not been taken up in this paper. A temporary passivity would be of no practical importance.

The use of lime in the trenches, so as to encase the pipe in an inhibiting alkali, ought to be of service, and is worth trying. Encasing in concrete would be expensive, but would protect the pipe, if properly done.*

Another method, suggested at Perth Amboy, is to leave the pipe in open trench. This might help materially, as the alternate wetting and drying would probably be less destructive than the constant exposure to moist air, salt, and carbonic acid in the swamp or meadow soil. At Atlantic City this principle has been carried one step farther, and the new 48-in., cast-iron, water main across the salt marsh has been raised above the surface of the meadow, so as to be entirely in the air.

As will be seen by Fig. 24, it is supported on reinforced concrete bolsters, 5 ft. 2 in. long, 20 in. thick, and about 5 ft. 6 in. high, so that the bottom of the pipe is about 1 ft. above the meadow. Each bolster, in turn, is carried on two piles of an average length of 37 ft., driven by a 2 000-lb. hammer, the final penetration required being not more than 1 in. under five strokes of the hammer falling 14 ft. The bolsters are 6 ft. from center to center. Between the pipe and the bolster there is a piece of hard vulcanized fiber, ⅜ in. thick. On the pipe laid in summer, expansion was provided for by allowing a minimum space of ¼ in. between the shoulder and spigot end of each pipe. In addition, a sleeve is inserted once in each 2 000 lin. ft. of pipe. The entire design has been carefully studied and worked out by Mr. Lincoln Van Gilder,† Chief Engineer, and T. Chalkley Hatton, M. Am. Soc. C. E., Consulting Engineer of the Water Department. Several thousand feet of the main are already completed. It would seem to solve the problem, either in a warm climate or where the main is large enough to be exposed without danger of freezing; and where it is permissible to have the pipe above the surface.

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* For use of cement as a wash, see Franklin Riffle, M. Am. Soc. C. E., Discussion on "The Preservation of Materials of Construction," *Transactions*, Am. Soc. C. E., Vol. L, p. 313 *et seq.* See also Discussion on "Relative Permanency of Steel and Masonry Construction," *Transactions*, Am. Soc. C. E., Vol. XLIX, p. 74, for many interesting data.

† Who furnished this information to the writer. See also *Engineering News*, Vol. LXX, p. 1046.

At Elizabeth an analogous, though less elaborate, expedient has been adopted. A 6-in. pipe could not be raised above the meadow and left exposed without danger of freezing, so it was laid on the surface, covered with a mound of meadow soil or mud some 6 in. thick, and the mound was then sodded with fox grass. This was done in the spring, so that the sod would make vigorous growth and protect the embankment. Although this gives great promise of obviating the trouble, it has not been in place long enough to predict its success with absolute certainty.

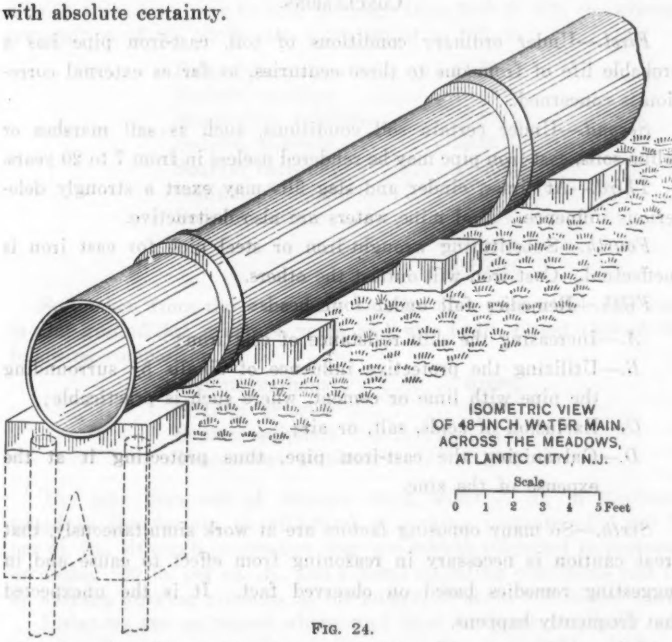

FIG. 24.

Very frequently, conditions are met which prohibit such a solution of the problem. At Syracuse, for example, where the mains are laid in the streets, such a mode of construction is not to be thought of.

Bearing in mind the protective effect of zinc, as shown in Fig. 11, it might at times be desirable to galvanize the cast iron in such locations, instead of giving it the ordinary bituminous coating. The United States Government has utilized galvanizing in some instances for the protection of gratings, covers, and similar castings, but as far as

the writer is aware, it has never been applied to cast-iron pipe. Double galvanizing, as it is called, should be used in such cases. This is a trade term, which does not mean that the iron has been dipped twice, but merely that it carries a much heavier coating than the ordinary galvanized metal. The heavy coating is not suitable for articles which may be subjected to bending, as it will crack off, but this objection does not apply to a rigid article like cast-iron pipe.

CONCLUSIONS.

First.—Under ordinary conditions of soil, cast-iron pipe has a probable life of from one to three centuries, as far as external corrosion is concerned.

Second.—Under certain soil conditions, such as salt marshes or saline soils, cast-iron pipe may be rendered useless in from 7 to 20 years.

Third.—At times, cinder and slag fills may exert a strongly deleterious influence. Acid mine waters are also destructive.

Fourth.—Substituting wrought-iron or steel pipe for cast iron is ineffectual. Cast iron will outlast the others.

Fifth.—Remedies fall under four heads:

A.—Increasing the skin resistance of cast iron;

B.—Utilizing the protective influence of alkalis by surrounding the pipe with lime or cement, where such is practicable;

C.—Exclusion of acids, salt, or air;

D.—Galvanizing the cast-iron pipe, thus protecting it at the expense of the zinc.

Sixth.—So many opposing factors are at work simultaneously, that great caution is necessary in reasoning from effect to cause and in suggesting remedies based on observed fact. It is the unexpected that frequently happens.

Acknowledgments.—The writer is under great obligations to the persons who have aided him with information, and to the published articles bearing on the subject. The references in the text will indicate to whom credit is due for this assistance, and to these gentlemen he wishes to express his thanks.

Doubtless much that is pertinent to the subject has not been mentioned, but it is hoped that any such facts may be brought out in the discussion.

DISCUSSION

Mr. Bowie.

C. P. Bowie,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest. Although the author has dealt primarily with cast-iron pipe, some of the methods for the prevention of external corrosion would apply equally to steel pipes. In California a number of companies are operating 8 and 10-in. oil pipe lines which pass through many miles of semi-arid lands where the soils are strongly impregnated with alkali. The total salts in these soils may be said to range from 0.05 to 5%, the average composition of the alkali salts being about as follows:

Potassium sulphate	11.28%
Sodium sulphate	19.48%
Magnesium sulphate	2.89%
Sodium chloride	23.03%
Sodium carbonate	40.96%
Sodium phosphate	2.08%
Sodium nitrate	0.28%
	100.00%

Soil taken from the immediate vicinity of a pipe which had been in the ground for nearly 4 years and was badly pitted showed the following composition:

Sodium carbonate	0.02%
Sodium chloride	0.05%
Sodium sulphate	1.31%
Magnesium sulphate	0.14%

The pipe lines are of wrought steel, about $\frac{5}{16}$ in. in thickness, tested in the factory to withstand a working pressure of 1 200 lb. per sq. in., although in practice they are seldom subjected to pressures exceeding 800 lb. per sq. in., and general operating conditions call for an average working pressure, at the pumps, of about 600 lb. per sq. in.

Instances are on record where such pipe, although originally protected with three coats of asphaltic paint, has corroded so badly in 3 years that the pit holes extended entirely through the metal. It is interesting to note that these pit holes often occur in portions of the pipe where the surrounding area shows little if any sign of corrosion. They are usually from $\frac{1}{2}$ to $\frac{3}{4}$ in. in diameter at the top, and often have the appearance of having been gouged out of the metal. Fig. 25 shows a number of such pittings in an 8-in. pipe which had been treated with two coats of asphaltic paint when laid, and had been in the ground a little more than 4 years. Fig. 26

* San Francisco, Cal.

Mr. Bowie. shows a cast-iron saddle used for repairing pit holes which have developed into leaks.

Although in some cases adjoining property owners would have no objection if the pipe lines were carried on trestles above the surface of the ground, as Mr. Pugh states was done with the 48-in. pipe at Atlantic City, this method causes serious difficulty when it is considered that oil is pumped through these lines at temperatures (at the pumps) as high as 160° Fahr., and that insulation against the loss of this heat, such as is afforded by an earth covering of 2 ft. or more, is often essential to their successful operation, especially during the winter. Where the lines are placed on trestles of any extended length, the necessary insulation must be supplied, and this is expensive, even under the most favorable conditions. It is worthy of mention in passing that in places where a continuous length of a mile or more of pipe has been carried on trestles, across low swampy lands abounding in bulrush or tule growth, an almost ideal insulation has been effected by enclosing the line in a boxing of 2-in. plank, the space between the inner walls of the box and the pipe being stuffed with the dried tule.

In reference to the use of concrete as a protection: this method was tried a number of years ago by one of the large companies, but without success. The expansion and contraction of the pipe, caused by the high temperature of the oil and the relatively low temperature attained during the occasional necessary shut-downs, which allowed the pipe to cool to the temperature of the ground, soon cracked the concrete covering so badly that it was rendered practically useless. Moreover, collar leaks, which are of frequent occurrence in such lines, were not only difficult to repair, but as they necessitated the cutting away of the old concrete at such points, presented an even more serious defect, in that it was found practically impossible to put on patches of concrete without having cracks develop between the old and new material.

A successful covering for such lines should be so elastic that it would expand and contract without rupture, in conformity with the pipe; it should afford insulation against loss of heat, and at the same time be impervious to the passage of water. Working along these lines, a number of coverings have been brought forward. Various kinds of asphaltum paint have been used, but, thus far, at least, without any marked degree of success. Crude oil has also been applied, by covering the pipe, after it has been lowered into the trench, to a depth of 6 in. or more with the oil, and then immediately filling in the trench with earth. This method is only partly successful, because, if the soil is at all sandy, the oil soaks into it and in time almost entirely disappears from the vicinity of the pipe, leaving the metal quite as badly exposed to the alkali as before the application.

FIG. 25.—PIT HOLES IN 8-INCH STEEL PIPE, DEVELOPED IN A LITTLE MORE THAN FOUR YEARS.

FIG. 26.—CAST-IRON SADDLE USED TO STOP LEAKS DUE TO PIT HOLES.

FIG. 19.—[illegible]

FIG. 20.—[illegible]

Quick-setting bituminous enamels, which can be put on in a layer from $\frac{1}{8}$ to $\frac{1}{4}$ in. thick, bid fair to give results. However, up to the present, these enamels have one serious drawback, in that they cannot be applied successfully to a moist pipe or to one having a temperature lower than 60° Fahr., because the enamel, although it is put on at a temperature of approximately 200° Fahr., is chilled so rapidly that it hardens before it can be spread. The action of the mop with which it is applied is merely to roll it into a myriad of balls or globules, which do not stick to the pipe but fall to the ground. This not only causes a great waste of material, but is apt to leave many small portions of the pipe, especially on the under side, wholly unprotected, even though several coats of enamel are put on. This difficulty, however, can be largely overcome by applying the enamel in warm dry weather only, or to pipes through which warm oil is being pumped. There are no lines in the State which have been covered by this process and allowed to remain in the ground for any great length of time. One line, passing through slightly alkaline territory, which was covered in this manner about 4 years ago, was found to be in excellent condition on recent examination. Mr. Bowie.

The method of covering most extensively used, and perhaps the most successful one, is that utilizing specially prepared roofing papers. Those commonly used are made from ordinary deadening felt; they are run through a mill in which the felt is plunged into a number of successive baths of hot asphaltum and rolled hot and under pressure after each bath until the fibers of the paper are thoroughly impregnated with the asphaltum. Just before entering the last set of rolls, the paper is sprinkled with either mica or soapstone. These minerals, though no doubt adding somewhat to resistance against the attack of chemicals, have for their principal function the quality of keeping the paper from sticking in the roll in hot weather. Asbestos papers, prepared in a manner somewhat similar, have also been used in a number of places. These papers are applied to the pipe by one of two methods known locally as the "spiral wrap" and the "longitudinal lap". The spiral wrap consists of applying the paper to the joints between collars by wrapping it around the pipe spirally. For this purpose, the paper is cut in the mill into rolls of the desired size, varying from 3 to 12 in. in width, and from 50 to 100 ft. in length, as best suits the diameter of pipe to be covered. The pipe is covered immediately after being screwed together by the tong gang, and while still on skids over the trench. It is coated with hot asphaltum, and the wider strips are wound on spirally before the asphaltum has had time to set. The crack which is left between each wrap of the wide strip is then coated with hot asphaltum, and a batten, or 3-in. strip, is wound on to cover it. The asphaltum is a thin grade of the ordinary refined product. It is applied to the pipe at a

Mr. Bowie. temperature of about 200° Fahr., and is of such consistency that it will not set for about 5 min. after application. If, due to climatic conditions or variation in the different shipments, the asphaltum sets too quickly, a so-called flux, which is simply a very thin grade of asphaltum, is added. It is usually the practice to ship 1 bbl. of flux to the field for every 60 bbl. of asphaltum. At the joints 3-in. strips stuck together with asphaltum are wound around the pipe on each side of the collar until a shoulder is built up flush with the outer circumference of the collar. The sleeve and shoulder thus built up are then coated with hot asphaltum, and the whole is covered with a 12-in. strip of paper. Besides being stuck to the pipe with asphaltum, this last strip is bound on with wire, as are also the ends of the paper where it is wrapped spirally. If this is not done, the ends will curl up and allow dirt to get between the paper and the pipe after the pipe has been lowered into the trench.

The longitudinal lap system, which, up to the present time, has been used much more extensively than the spiral wrap, consists (as the name implies) of wrapping the paper longitudinally around the pipe. The paper is delivered in the field in rolls, about 6 in. wider than the circumference of the pipe to be covered, each roll containing approximately 72 lin. ft. Two men go ahead of the paper gang and measure up each joint of pipe between collars and cut the paper into the desired lengths.

In applying the paper to the pipe undoubtedly the best method is first to coat the pipe with hot asphaltum, as previously described, then to wrap the sheet of paper around the pipe and stick down the lap with hot asphaltum. This, however, is not often done when covering lines where the oil passing through is to be heated, the supposition being that the heat of the oil (usually about 130° Fahr.), combined with the pressure of the earth on the pipe after it has been buried, will be sufficient to soften whatever asphaltum there is in the paper and form the necessary bond between paper and pipe to keep the water out. Without question, this supposition is correct to a certain extent. The paper, especially where a heavy quality is used, will undoubtedly adhere to the pipe in most places where it comes in contact with the metal; but it is to be noted that roofing papers applied to a pipe, while the latter is still on skids above the ditch, and allowed to stand in the sun for a few hours, will stretch to such an extent that the contact will be found to be on the upper part only, there being often as much as ¼ in. of space between the bottom of the pipe and the paper. It is a serious question if this does not occur when the heat from the oil first reaches the paper, even though the pipe has been lowered into the trench and covered up. The paper, of course, does not bag uniformly, as in the case above ground, but the surplus is taken up in wrinkles, and these wrinkles

Mr. Bowie.

are very apt to be connected for the entire length of a joint of pipe. Should an injury occur to one of these wrinkles or a crack develop in it, which will undoubtedly be the case as oxidation goes on, it is not at all improbable that water, which would enter, would reach every place in a joint of pipe where the paper was not securely stuck on, and thus render the whole covering practically useless. Where asphaltum is used as a binder, the wrinkles that form will be filled, thus preventing the water from entering, even though, in time, the paper should crack at these points. Time alone, however, can solve this much-mooted question, as there are no lines in California protected in this manner which have been laid a sufficient length of time for these cracks to develop.

For covering the collars, the same method is used as with the spiral wrap, except that the binder is often omitted. This, it would seem, is even more serious than omitting the binder from the main body of the pipe, as it gives the water a chance to get between the paper and the pipe through the ends of the wrinkles.

Whether or not a binder is used, the lap should always be made on the top of the pipe with the outer portion of the lap downward, and the wires should be placed about 16 in. apart. This is necessary in order to prevent the paper from being pulled away from the pipe by its own weight before it can be lowered into the ditch and covered.

Mr. Allen.

KENNETH ALLEN,* M. AM. SOC. C. E.—With respect to the Atlantic City pipe, there are two old cast-iron mains, one 6-in., laid in 1881, and one 20-in. laid about 6 years later. Between 1902 and 1905, when the speaker was there, these pipes had deteriorated greatly, as described by Mr. Pugh. Soft spots in the metal were frequent, and although the pipes were in service and held the pressure, it was felt that it was not good policy to depend on them. In places the metal could be whittled like chalk, and yet no failure occurred in the 6-in. pipe when it was tested with a pressure of 75 lb. per sq. in.

With regard to the 40-in. steel main, which was laid in 1901 beside the cast-iron mains: that pipe, which was constructed of ¼-in. open-hearth plates, corroded much more rapidly. Two small perforations were discovered on February 2d, 1905; fourteen by September 1st, and twenty-two by January 1st, 1906. On September 1st, 1905, the speaker estimated that it would cost $345 340 to duplicate the three mains, and that their actual value then was $187 100, showing a deterioration of 46 per cent.

An inspector with a kit of tools walked the length of the mains from the pumping station and back—about 12 miles—every day; and when he found a hole in the steel main, he would whittle a small stick and plug it up—a very simple and effective expedient.

It seemed to the speaker at the time that, in view of the cost of the

* New York City.

Mr. Allen. pipe—about $200 000—it would be desirable to preserve it, if possible, by a casing of 3½-in. reinforced concrete. This, it was estimated, would cost about $58 000, or $11 500 per mile, where exposed to the meadow conditions. Before deciding on this, however, it was concluded to make a test by covering in this way several short sections containing leaks, a contract for which was let before the speaker left. Later, however, it was decided to abandon this main altogether, and to put in a 42-in. wood-stave pipe. Still more recently, the 48-in. cast-iron pipe, which Mr. Pugh has referred to, has been constructed. Cast-iron has always been very difficult to handle on the meadows and, on account of vibrations from railway trains near-by, difficult to maintain in a tight condition; but it was believed that it would be much more durable than steel and, if raised above the surface of the meadows on cradles, would be more satisfactory on the whole than wood-stave pipe.

The author states that the "soil was free from organic acids". It is understood that analyses made by W. P. Mason, M. Am. Soc. C. E., some years ago, indicated the presence of vegetable acids which, of course, would promote corrosion of the iron. This action may have been increased at certain points by electrolysis, as tests, made in July, 1905, indicated a fairly constant flow of 30 amperes through the two cast-iron and the 30-in. steel mains toward the mainland where the current passed off to street railway tracks and a local water main. Whether the alteration in the metal referred to by the author may have been due partly to this cause is not known, but is suggested as possible.

The winter of 1904-05 was one of great severity, and the 12-in. main froze for the first time, breaking twenty-two lengths of pipe. It seems to the speaker quite probable that the loss would have been less if the pipe had been new and sound.

In the Annual Report of the Water Department dated August 1st, 1901, W. C. Hawley, M. Am. Soc. C. E., then Engineer and Superintendent, stated that:

"The cast-iron of the pipe has been changed in many places to a condition such that it can be cut by a knife, like chalk or graphite, and its specific gravity is only about 2.3. Some cases have been found where this condition extends entirely through the shell of the pipe; in others it extends from one-eighth to three-eighths of an inch in depth. This deterioration is not uniform in the same pipe; some of the pipes, in which the metal has in places changed through the entire thickness, having other places where no perceptible change has taken place. In some spots the tar coating is still to be found, but for much of the outer surface of the pipe, it has entirely disappeared."

Later, when the speaker succeeded Mr. Hawley in office, he found the conditions much the same, although no point was observed where this alteration extended through the entire thickness of the pipe. From

a general consideration of these facts, the speaker would estimate 25 years as the extreme limit of the time during which dependence could be placed on cast-iron pipe under similar conditions, and is of the opinion that steel pipe coated as this was, with two layers of burlap and mineral rubber, is not suitable for use. Mr. Allen.

It may be of interest to add that while a sewer trench was being excavated along the water-front in Baltimore, several cannon balls varying from 4 to 6 in. in diameter were found several feet below tide-level, and, as there had formerly been a landing at this point, which subsequently was filled in, it seemed probable that these had been dropped from some vessel during the War of 1812. In spite of having remained in this position so many years, they were practically free from rust.

WILLIAM W. BRUSH,* M. AM. SOC. C. E.—New York City has more than 2 500 miles of pipe in its distribution system, and has had practically no difficulty from external corrosion. An examination, made about a year ago, of a length of 48-in. pipe which was laid in 1867 in Central Park, near the large reservoir, in ground that was low-lying and wet a good part of the year, showed the outside coating to be practically perfect. The general condition of the entire pipe system of New York City is such that the outside of the pipe generally shows no corrosion that would affect its life. Mr. Brush.

The interior of the pipe shows very materially the difference in effect of waters supplied to the different boroughs of New York City. Croton water has such slight corrosive effect on cast-iron pipe that sections of mains laid shortly after this water supply was introduced, in the early Forties, or in use some 60 years, show practically no corrosion. Therefore, there has been no necessity for cleaning these mains; whereas in Brooklyn, where the water has a comparatively high chlorine content, varying during the past 50 years from a maximum of about 25 parts per million to a minimum of about 8 parts, with an average for the past 20 years of possibly 15 parts per million, there is a corrosive or tuberculating effect, especially on pipe laid uncoated. The pipe laid in 1858 was mainly Scotch pipe, in 9-ft. lengths, and was uncoated. There was also some pipe cast in the United States at about that time and laid uncoated. Such pipe required cleaning, as the tubercles in a 6-in. pipe were of such size that a 2-in. rod could not be passed through the pipe without breaking off some of them. That pipe, when cleaned, was good, apparently, for another 50 years, as far as the strength of the wall limited the life. The rusting probably advances rapidly on the cleaned pipe. Thus, at Far Rockaway, pipe which had been laid from 20 to 30 years ago, and was cleaned about 10 years ago, now shows tubercles of a depth and volume equal to those found in pipe laid some 20 or 30 years

* New York City.

Mr. Brush.

ago, which has not been cleaned. Apparently, with that particular water, tubercles developed in 10 years after cleaning, which would have taken 20 or 30 years to develop in the pipe as originally laid.

It is very important, however, to determine under what conditions rapid corrosion of the iron will take place on the exterior of the pipe. In many cases, even where protection would cost large sums, the cost of external protective measures for pipe lines would pay. On the other hand, there are cases where such protection would not pay.

Experience in New York City, where the pipes are frequently laid in ground-waters, especially along the water-front, indicates that there is no condition that would warrant any special method of protection.

Mr. Wagner.

SAMUEL TOBIAS WAGNER,* M. AM. SOC. C. E.—The author has referred to pipe laid in Philadelphia in 1817. In 1897, about 1 000 ft. of this pipe, 22 in. in diameter, was removed from the bed of Pennsylvania Avenue on account of street changes. After 80 years of service, its external condition was almost perfect. It had been laid in a sandy and fine gravelly soil, on ground well above tide-water. The full section of the metal in the pipe was there, and although tubercles were rather freely distributed on the inside, the loss of waterway section was only about 4.2 per cent. The metal inside was generally in excellent condition.

Reference has been made to the effect of chlorine in the water on the internal condition of a cast-iron pipe. This is a new thought to the speaker, and for information, it might be interesting to add that the water in the Schuylkill River, which passed through this pipe line, has a chlorine content of about 5 to 6 parts per million.

The data presented by the author are of great interest, and serve to accentuate the necessity of studying local conditions before using any metal when corrosion is to be feared. There are many places where cast iron is the very best material that can be used to resist corrosion, and it would appear that the author believes that water pipe, when not exposed to the action of salt water, is one of them. Some cases of the resistance of this metal to ordinary atmospheric conditions are remarkable. In some of these, cast iron is still perfect where wrought iron and steel, similarly exposed, have rusted away. There can be no doubt that the resistance of the skin coat of silicious material is one of the chief reasons for its durability.

Mr. Kellogg.

R. C. KELLOGG,† JUN. AM. SOC. C. E.—There has been so much advertisement and so much written in regard to the everlasting life of cast-iron pipe that information such as contained in this paper is of value, in order that the exceptions to the general rule as to the longevity of such pipe can be taken into consideration when using it.

* Philadelphia, Pa.

† Brooklyn, N. Y.

Some years ago the speaker was familiar with conditions of quite severe pitting on a 60-in. riveted, steel main laid from Astoria to Ravenswood, Queens County, New York, to convey gas from the new Astoria Works to the Ravenswood plant. At the time, these pittings were attributed to electrolytic disturbances, but, in view of the information presented in this paper, it is only fair to assume that this was an error, and that the major causes were tidal action and the soil in which the main was laid, which was originally part of a salt marsh. This idea is somewhat strengthened when the speaker recalls the fact that an electrolysis survey did not seem to develop sufficient evidence that electrolysis was the sole cause of the trouble. He is familiar with several cases in Brooklyn where cast-iron gas mains have been taken up after having been in the ground from 50 to 60 years, such mains showing no signs of exterior corrosive injury. There are also several cast-iron gas mains crossing the Harlem River, at various points, all of which are covered by salt water and mud. Parts of these mains have been taken up, on account of accidental breakage, and show absolutely no effect of corrosive action. Mr. Kellogg.

From this it would appear that corrosion is caused, not only by soil of a certain character, and tide-water, but also that the air has an appreciable effect. The speaker is not familiar with any cases of internal corrosion, because his experience has been entirely with gas pipes, in which the interior surfaces are preserved by the deposition of liquid hydro-carbon compounds condensed out of the gas, due to friction and changes in temperature.

A. D. FLINN,* M. AM. SOC. C. E.—Some years ago, while visiting one of the yards of the Spring Valley Water Company, in San Francisco, the speaker was shown several lengths of wrought-iron pipe which had been taken from the ground after 20 years' service and was in perfect condition, both inside and outside. The pipe had been made in accordance with specifications by the Chief Engineer of the Company, and the asphalt coating had been made and applied also according to these specifications, the work being done with great care in all details. The speaker was informed by the engineer who accompanied him that pipe was frequently taken up, where changes in the system were necessary, and found to be in such excellent condition that, when necessary, it was re-used. Mr. Flinn.

In connection with the Catskill Aqueduct, in laying pipe across The Narrows, from Brooklyn to Staten Island, a search was made for a coating that would be a little more durable than most of the known pipe coatings, and some inquiries gave hopeful accounts of one known in the trade as "bitumastic enamel". The pipe line, on

* New York City.

Mr. Flinn. which this coating is being used, is now being laid, and the work will probably continue during the fall and next spring.

In respect to galvanizing as a means of protecting steel and iron, experience on the Catskill Aqueduct is encouraging. Steel-plate manhole covers, railings, and other objects which have been galvanized and exposed to the weather, have remained in good condition.

Mr. Boucher. WILLIAM J. BOUCHER,* ASSOC. M. AM. SOC. C. E.—The use of cast-iron pipe is so general that the speaker believes that any data relating to the subject of corrosion will be of interest, perhaps of value.

Until 1897 or thereabouts, Jersey City, N. J., was supplied with water from the Passaic River. The water was pumped from a station at Belleville, opposite North Newark, into and through a 36-in. cast-iron main laid alongside the Turnpike Road to a reservoir on Bergen Hill, Jersey City. This pipe was made by the Warren Foundry and Machine Company, and the letters, "W. F. & M. C., 1862", are still discernible to any one with good eyesight. The pipe is supported, as described later, practically at the surface of the grass-and-weed-covered tidal salt marsh, which extends from the west side of Bergen Hill about 7 miles to the east bank of the Passaic River. It was laid during 1862 and 1863 and, with the exception of a few months in 1898, has conveyed water throughout its whole length since 1863. It shows no signs of pitting or disintegration, and is in practically a perfect state of preservation. With a pocket knife, the speaker removed a few patches of covering resembling asphalt. These patches covered only a minute fraction of the entire surface, and if the pipe was at one time entirely covered with a preservative, it has long since ceased to be of any value, as the pipe, everywhere, is uncovered and the upper portion of its circumference plainly shows that it is used frequently as a foot-path.

The following description of the construction, etc., taken from the "Report of the Engineer of the Board of Water Commissioners, 1862-63," may be of interest:

"Two rows of piles are driven, three feet apart, to a solid foundation, which is found at a depth varying from 12 to 20 feet below the meadow surface. The heads of the piles are then cut off 16 inches below the meadow and cross capped with 10 x 12-in. pine timber; upon this capping a brick pier 12 inches thick, 44 inches long, and 13 inches high, is laid up in hydraulic cement.

"The pipe when placed upon this pier is securely held in its position by additional courses of brick work built up on its sides. The piling and capping are thus kept constantly immersed in water, rendering it perfectly secure from decay, and the joints of the pipe being just above the meadow surface enable them to be caulked without difficulty."

* New York City.

The pipes are 36 in. in diameter, $1\frac{1}{20}$ in. thick, and 12 ft. 5 in. long, averaging 4 800 lb. in weight. During 1862, 22 400 lin. ft. of completed main were laid, at a cost of: Mr. Boucher.

Iron pipes at $37 per ton, delivered...............	$138 121.30
Contract for laying pipes, including piling, capping, brick foundations, and covering pipes with earth.	46 567.20

The object of the earth covering was to prevent the freezing of the water during the winter, but, according to the speaker's recollection, the pipe has had no earth covering for 20 years, and was conveying water when observed in October, 1914.

GEORGE M. PURVER,* ASSOC. M. AM. SOC. C. E.—The speaker would like to ask Mr. Pugh, if he had any opportunity to observe the rate at which rusting of cast-iron pipe takes place. From the time a pipe begins to rust, does the rate of rusting remain uniform and vary directly with the time of exposure, or not; and would the author expect the same results under similar conditions? Mr. Purver.

The speaker believes that this paper should prove interesting and instructive to every member of the Profession, and it has suggested to him the idea that it would be desirable to urge the Society to appoint a "Special Committee on Deterioration of Structural Materials," such as stone, concrete, steel, cast iron, and wood.

Such a committee could gather all the data available on rusting of metals, rotting of wood, disintegration of stone and concrete, etc., and plot curves showing periods of time of their reliable service. It is possible that, with the assistance of coefficients established from observation of the conditions surrounding the material in use, one would be able to approximate the rate of deterioration.

The speaker does not recollect a single volume or pamphlet on this important subject that would comprise all the reliable data on the life of various structural materials, and yet in preparing an estimate or in evaluating an engineering enterprise, such data would furnish an important element.

W. J. E. BINNIE,† ESQ. (by letter).—The author is to be congratulated on his exceedingly interesting and most useful paper. The subject of corrosion of metal pipes is one of great interest to all water engineers, and in the past too little attention has been paid to the study of the character of the various soils in which pipes are to be laid, with the consequence that their life has sometimes been lamentably short. Mr. Binnie.

It is generally considered that, owing to the foundry skin, cast-iron pipes do not corrode so rapidly as steel. It is very seldom, however, that even a cast-iron pipe, when taken up after being several

* Brooklyn, N. Y.

† London, S. W., England.

Mr. Binnie. years in the ground, does not show external corrosion, to a certain extent, thus pointing to the fact that the usual coating materials do not afford permanent protection.

Fortunately, the corrosion is generally confined to a certain depth, that is, pitting occurs, accompanied by nodular incrustation, which, at first local, spreads in area as time goes on, so that the entire surface may eventually become corroded; but this pitting, whether in cast-iron or steel pipes, seldom extends to a depth of more than ⅛ in.; and when the surface is entirely covered with nodular incrustation, a stage is reached when further deterioration of the metal ceases.

With cast-iron water pipes, the factor of safety is large, and the necessary minimum thickness for good castings is considerable, with the consequence that the pipe is still strong enough, even when attacked by ordinary corrosion both within and without, though its carrying capacity may be affected seriously. Special cases occur, however, such as those which the author brings forward, when the material of the pipe itself is entirely changed without loss of outward form, and the pipe becomes useless; and papers such as this are of great service in showing the engineer what is to be expected if he lays pipes in similar ground without taking special precautions.

The experience of the writer's firm leads them to believe that the depth of pitting does not differ much, whether the material is cast iron or steel, but the loss of ¼ in., due to internal and external corrosion together, will be of very serious consequence in the case of steel pipes.

The writer's firm was engaged by the Western Australian Government, in conjunction with Sir W. Ramsay, and Mr. Otto Hehner, to report on the remedial measures which should be adopted to arrest the corrosion which was taking place in the pumping main for the supply of water to the Western Australian Goldfields. This main was 350 miles in length and 30 in. in diameter. It was corroded internally, owing to the faulty coating and the character of the water, so that some sections had lost 50% of their carrying capacity; and externally to such an extent that the main was sometimes completely penetrated.

The external corrosion was most active in saline and in clay soils, but where the main was laid through sandy light soil, corrosion was generally slight. The remedial measures which were undertaken to deal with the external corrosion where the pipe had to be underground consisted in opening up the trench, cleaning and scraping the outside of the main, re-coating, and wrapping it with tarred Hessian. In addition to this, it was suggested that the pipe should rest on a bed of lime concrete, and should be surrounded with a layer of slaked lime when the trench was refilled, but it is not known whether or not this was actually done.

The writer felt fairly confident that the tarred Hessian, if perfect, would in itself be a sufficient protection, but it was not at all an easy matter to wrap this Hessian so that it would adhere to the entire surface of a lock-bar jointed steel pipe, such as the Coolgardie main. Mr. Binnie.

To remedy the internal corrosion, the writer suggested the addition of lime to the water, and also the removal of the dissolved air, as experiments had shown how easily this could be done and how slowly corrosion took place with this air-free water. It is believed that the Western Australian Government has carried out the first recommendation, but, as far as known, it has not tried the latter.

The writer's firm is now laying a steel pipe line from Wales across the estuary of the River Dee to Birkenhead, in Cheshire. The pipe, where it crosses the estuary, is in salt marshes slightly below high-tide level for several miles, and owing to the high pressure and other reasons it was desirable to use steel pipes. These are flanged pipes, and, owing to the restrictions placed on the Birkenhead Corporation, had to be laid underground.

The writer's firm has decided to take the following precautions:

(1) To wrap the pipes externally with two layers of Hessian treated with a bituminous solution similar in composition to that used for cables.

(2) To lay the pipe in lime concrete except at the flanges, which will be surrounded with puddled clay mixed with slaked lime in the proportion of 6 parts clay to 1 part slaked lime, in order to facilitate the replacement of a faulty pipe.

LEONARD S. DOTEN,* M. AM. SOC. C. E. (by letter).—The writer has had considerable experience in the use of cast iron in sea water, chiefly in the form of water mains and bearing piles for wharves. As a result of this experience, he concludes that cast iron is much superior to steel or wrought iron in resistance to corrosion. Mr. Doten.

The author states:

"Five separate theories have been advanced to account for the rusting of iron, but, at present, three of these have been practically eliminated, leaving only two over which the fight still wages: the acid theory and the electrolytic theory."

In the majority of cases corrosion can be explained most satisfactorily by the latter theory, but no one theory accounts for it under all conditions; corrosion may be either the result of chemical action or electro-chemical action, depending on controlling conditions. The cases of rapid disintegration of water pipes cited by the author were undoubtedly examples of electro-chemical action. The writer does

* Washington, D. C.

Mr. Doten.

not mean to imply that the electrolytic theory, as usually understood, accounts for the corrosion satisfactorily in those cases.

On page 812 the author states that "the salt of the ocean is particularly destructive to cast iron, * * *." The writer believes this to be true only under certain unusual conditions, which will be described later. It is a well-known fact that the superstructures of many of the wharves along the Atlantic Coast are supported by cast-iron piles. Cast iron is the most durable material, available for use, for wharf piling in salt or fresh water. The Government pier at Old Point Comfort, Va., is a good example of this type of construction. This wharf was built about 32 years ago. The writer knows from personal inspection that the piles of this pier are now in practically as good condition as on the date of construction. The steel I-beams resting on these piles have been much weakened by corrosion, scale having formed on them ½ in. or more in thickness.

In 1903, the writer had a very interesting experience, in investigating the cause of the rapid disintegration of a submarine water main and in devising a means to prevent further trouble from this source. The pipe line in question was in Boston Harbor, and connected one of the distributing mains of the Boston Water-Works with the distributing system of one of the forts. The line was approximately 2 000 ft. long. It was of heavy, flexible-joint, coated, cast-iron pipes, 6 in. in diameter, and was laid in a trench 5 ft. deep, and covered with clay, in accordance with the standard practice in that locality. Near the shore of the reservation, a break had occurred which was the third at this point, due to the same cause, since the construction of the line, about 7 years previously.

When removed, it was found that there were several holes in the bottom and sides of the pipe, ranging in diameter from ½ in. to 1 in. Aside from these holes, the general appearance of the pipe was good, and the coating apparently was intact. By very close examination, however, it was possible to discover many small circular areas where the coating was elevated slightly above the general surface. At these points, it was possible to cut deep holes with a knife, in some cases extending completely through the shell of the pipe. The appearance of the disintegrated metal was similar to that of graphite, which is characteristic of cast iron which has undergone the change produced by electrolysis. On exposure to the air for a time, the substance became harder.

On account of the close resemblance of the corroded pipe to that which had been injured by electrolysis, and the fact that the line affected was at a place where injury due to this cause would be most likely, a test was made to ascertain whether an electric current was passing through the line, but gave a negative result.

Following this, another theory to account for the corrosion came to the writer's mind. It had been noted that there was a large accumulation of finely divided anthracite coal on the beach in the immediate vicinity of this line. This coal had been brought there by the action of the tidal currents from the opposite shore of the island, where a coal barge had been wrecked several years before, and had come into contact with the pipe in the trench. Under the pipe there were large masses of cemented gravel having the color of iron rust. Mr. Doten.

Furthermore, each time a defective pipe had been replaced with a new one, it had been necessary to extend the trench a little farther, in order to make a suitable connection with the sound pipe. The small pieces of coal became distributed over the bottom of the trench to a greater or less extent during this operation, and, as a consequence, more pipes were affected. This probably accounts for the fact that only two lengths were removed the first time, whereas about eight lengths were removed the last time.

Taking these facts into account, the writer devised the theory that the action causing disintegration of the metal was electro-chemical, and resulted from the contact of the coal (carbon) with the iron in an electrolyte (sea water). An experiment was made to ascertain the correctness of the theory. A crude electrolytic cell was formed by using a large glass jar of sea water, a piece of the pipe metal, and a large piece of anthracite coal. Iron wires were used to connect the two poles. Connecting these wires to a galvanometer the writer obtained an initial deflection of 5° and a constant deflection of 3½° when the instrument was adjusted for the proper external resistance. Gas bubbles formed on the poles, in the manner characteristic of electrolytic action.

As a result of this experiment, the writer was convinced that the theory was correct and that the metal of the pipes had been carried away by electrolytic action resulting from local currents.

The report which was made to the War Department, dated February, 1904, contained the statement that, in the writer's opinion, the disintegration of the pipe was due to galvanic action between the coal and iron, and a recommendation that the new pipe as soon as placed be encased with a rich mixture of concrete. The work was carried out in accordance with the recommendation of that report. No further trouble due to this cause has been experienced since that date.

As electrolytic action is produced when carbon is placed in contact with such metals as zinc, tin, and iron in an electrolyte, it is the writer's opinion that all forms of coal, peat, and other vegetable material which has been more or less carbonized, as well as many other substances, also produce electro-chemical action under similar conditions. The rapid corrosion of the water pipe lines at Elizabeth, Perth Amboy, and Atlantic City, N. J., Syracuse, N. Y., and those lines laid

Mr. Doten. in cinders from power plants, could be accounted for most satisfactorily in this way.

Undoubtedly, cast-iron wharf piling and the steel or iron supports for structures in sea water have been weakened by such action. Therefore, the writer believes it would be unwise to use cast iron or steel in the substructures of coal wharves in such locations. Electrolytic action would be likely to take place at the bottom.

The author's statement that "to get the full corrosive effect of sea water, plenty of air is essential" does not well apply to cast-iron piles, as many of those which have been in place 30 years or more show little evidence of corrosion. It applies more accurately to wrought iron and steel. In the case cited by the author, the piles failed at the bottom, where the air was probably least plentiful.

It is also stated that the meadow soil at Atlantic City was free from organic acids. The soil conditions were suitable, and there was sufficient sodium chloride present to make electrolytic action possible. The more rapid corrosion of this pipe at the top than at the bottom could be accounted for by the accelerating effect of the sun heat of summer or by a difference in the character of the soil.

Mr. Pugh. MARSHALL R. PUGH,* M. AM. SOC. C. E. (by letter).—In a number of instances corrosion of steel pipe is closely linked with that of cast-iron pipe, is due to the same causes, and is to be prevented by similar methods. The conditions cited by Mr. Bowie in the California oil pipe lines are severe. The high pressure to which they are subjected and, still more, the great temperature variations, undoubtedly call for special treatment. The writer was much interested in the method of wrapping, which is quite similar to that recommended for protecting the Goldfields main in Western Australia.† In this case a 30-in. steel pumping main, 350 miles long, used to supply Coolgardie, was found to be rapidly failing from both internal and external corrosion. As the cost of the main was about $10 000 000, it was a matter of very great importance. It appeared on investigation that two of the recommendations of the commission of engineers had not been followed in its construction; first, that the pipe should be dipped in a nearly boiling asphaltic composition, both in England and immediately before being laid in Australia; and second, that the pipe be laid above ground. In reference to the first point, the Joint Report says:

"(1) If the pipes are dipped just before laying, the deterioration of the coating due to exposure or possible damage in transmit is minimised. (2) It has been found that a coating which appears perfectly satisfactory to the naked eye contains, when examined with a magnifying glass, small holes due to minute bubbles which penetrate right

* Philadelphia, Pa.

† "Goldfields Water Supply, Corrosion of Steel Main." Joint Report of Sir Alexander Binnie, Son and Deacon, Sir William Ramsay, F. R. S., and Mr. Otto Hehner, F. C. S.

through to the metal beneath. On removing one of the nodular incrustations which form, as a result of corrosion, the coating at first sight frequently appears to be sound. A more minute examination, however, reveals the fact that the metal has decayed below the coating, having been attacked by the penetration of the water through one of these minute holes to the metal surface. Were the pipe to be dipped a second time as recommended by the Commission, care being taken that the first coat remained uninjured, the odds against the occurrence of two air bubbles, one above the other, giving direct access of the water to the metal, would be enormous." Mr. Pugh.

The remedial measures recommended have been stated in the discussion by Mr. W. J. E. Binnie.

The writer would be interested in knowing the cost of wrapping pipe with roofing papers coated with asphaltum, and how it compares with the cost of using galvanized pipe.

The cast-iron saddle shown by Mr. Bowie, Fig. 26, is analogous to the more primitive expedient on the 30-in. steel main at Atlantic City, where the hole was closed with a wooden plug, and a box was strapped to the pipe and filled with concrete.

The expansion and contraction due to temperature variations in the oil pipe lines obviously makes concrete a poor material with which to protect them, but if surrounded by a lime and clay puddle, the desired result might be obtained. Whenever pipes are cased in concrete it would be of great advantage to adopt the expedient mentioned in the discussion relative to the Birkenhead pipe. In this instance the joints are not surrounded with concrete, but are buried in a puddle of clay and slaked lime. This enables a length of pipe to be removed without the necessity of cutting the concrete.

Mr. Allen, whose opinion should have great weight, estimates the extreme life of cast-iron pipe, under conditions obtaining at Atlantic City, as 25 years.

The question raised by Mr. Purver as to whether or not the rate of rusting remains uniform and varies directly with the time of exposure, is difficult to answer with any data in the writer's possession. It would appear reasonable to assume that it goes on at an accelerating rate, because both electrical and physical conditions would tend to promote more intense corrosion, at all events within certain limits. As bearing on this, the case of internal attack mentioned by Mr. Brush is of interest. He states that, at Far Rockaway, pipe laid from 20 to 30 years ago, and cleaned about 10 years ago, now shows tubercles of a depth and volume equal to those in pipe laid at the same time which had not been cleaned. Possibly this may be due in large part to the fact that a scraped pipe has the surface left in very favorable condition for corrosion, whereas the new pipe is protected for quite a number of years before the attacks can even begin.

Mr. Pugh.

For the purpose of observing the effect of skin resistance, the writer had a series of plates made, 1.5 by 2.0 by about 0.28 in. Some of these were of ordinary cast iron, and some were of iron cast in moulds the facing sand of which had been mixed with chromite, $FeCrO_4$, ground to fine powder. Three of the plain plates and three of the chromated plates were immersed in a glass containing 100 cu. cm. of a 3% solution of sea salt in water, which closely approximates the composition of sea water. The edges of all the plates were filed smooth and paraffined, so as to confine the corrosion to the two faces of the plates. The plain cast iron contained no chromium. Filings of the skin of the chromated plates contained 0.42% of chromium.

The results, given in Table 9, do not indicate any very promising outlook for such methods of protection.

TABLE 9.

No. of sample.	Kind of iron.	Weight of sample.	Weight, 10 days.	Weight, 54 days.	Weight, 5 months.	Loss of weight in 5 months.	Loss of weight per square inch of exposed surface.
1	Plain cast iron.	108.765	108.716	108.600	108.180	0.585	0.0975
2		97.511	....	....	96.888	0.623	0.104
3		94.307	....	....	93.793	0.514	0.0857
4	Chromated cast iron.	97.421	97.357	97.252	96.874	0.547	0.091
5		101.209	....	....	100.561	0.648	0.108
6		100.765	....	....	100.084	0.681	0.113

In corroboration of what Mr. Flinn has said about the protective effect of galvanizing on the Catskill Aqueduct work, the writer had an opportunity to ascertain the condition of a galvanized-iron ocean outfall sewer constructed under his supervision 11 years ago. This pipe was recently exposed at mean tide level near the shore, where it had constantly been subjected during that period to alternate action of air and water externally and sewage internally. It was a matter of comment, by all who saw it, how bright, new, and uninjured the galvanizing was. The pipe was apparently as good as the day it was laid, and the zinc still retained its bright, fresh appearance.

Mr. Doten has presented an interesting and unique case of cast-iron pipe corrosion, and his experiments would seem to establish the correctness of his conclusion that "the action causing the disintegration of the metal was electro-chemical, and resulted from the contact of the coal (carbon) with the iron in an electrolyte (sea water)." In a general way it is analogous to the instance cited by Krzizan, on page 817, and, taken in conjunction with that case, shows need for care in a number of what at first sight might appear to be harmless conditions. The writer, nevertheless, cannot go so far as to ascribe all such disin-

tegration to this source. For example, a case was brought to his attention by Mr. B. B. Hodgman, in which a peat bog of great depth, a mile or so from Belle Plaine, Iowa, was crossed by the Chicago and North Western Railway. The Railway Company about 1894 laid a 6-in. cast-iron water pipe, some 1 800 ft. long, from a pumping station to its roundhouse. It passed through this bog, in which it was originally laid from 18 to 20 in. deep, and in 10 years had sunk to a depth of about 4 ft. When examined by Mr. Hodgman in 1908 the pipe had been in place some 14 years. It was badly corroded externally, with pittings ¼ in. deep and 1½ in. in diameter. In this case there was no salt water to act as an electrolyte. Mr. Pugh.

A peat bog is rich in the humus acids; the ligneous fiber of the sphagnum mosses on decaying gives rise by a fermentive process to humic, ulmic, and other rather ill-defined acids. These acids are powerful agents in dissolving mineral matter from rocks, and even quartz may be corroded by them. Large deposits of bog-iron ore are found in such swamps. The corrosion of the Belle Plaine pipe would thus seem to be readily explained.

That coal may be an important contributing cause when cast-iron pipe laid in cinders is subjected to the action of salt water would appear probable, but there is also no question of the dominant rôle played by sulphur, as noted at the Harrisburg tunnel, on page 850.

Mr. T. N. Thomson* cites two cases of corrosion in wrought-iron and steel pipes. One, a piece of 1½-in. black wrought-iron exhaust pipe from power boilers in Tonawanda, N. Y., had lain in damp soil and ashes 4 years. No pitting was present, but the pipe varied from about half its original thickness down to that of tissue paper. The second, a 2-in. steel pipe, from Boston, was laid on top of a cement floor over which was a plank floor on which coal was stored. Some of this coal worked down through the planking so that the pipe was partly buried in coal. After each heavy rain, surface water leaked into the coal, so that it is probable the coal around the pipe was constantly moist. This pipe, laid in February, 1904, failed through external corrosion, and had to be removed a year later. It is scarcely safe to assume that this was due entirely to what Mr. Doten terms electro-chemical action. It is a well-known fact that the water leaching through coal becomes strongly acid from the sulphur in the iron pyrites found in the coal. The acid waters, leached from old culm banks in the coal regions, in many cases have destroyed every vestige of vegetable matter near them. These acid waters, as already stated in the paper, are very destructive of iron, and may have to do with the destruction of the substructure of coal wharves, cited by Mr. Doten.

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* *Journal*, Engrs. Soc. of Pennsylvania, Vol. 1, p. 233.

Mr. Pugh. The writer still adheres to his views regarding the effect of air on the rapidity of corrosion of different parts of the pipe, as noted on page 844 and illustrated by Fig. 22. This explains satisfactorily why, at Atlantic City, the pipe was perforated most frequently at the top, whereas at Elizabeth it deteriorated chiefly at the bottom. Nor is this inconsistent with the fact that pipe laid in open trench, but still subjected to alternate wetting and drying, lasts longer than pipe in a marsh, although that in open trench is exposed more freely to the air whenever the water falls. The moist, aerated soil forms just the condition for most vigorous action, whether chemical or electro-chemical. Furthermore, in the soil, the ferric hydroxide is retained mechanically in contact with the pipe in large quantity, as shown by the analysis in Table 7. Ferric hydroxide acts as a catalytic agent in hastening many chemical reactions, and may thus accelerate the corrosion of the pipe.

Soil conditions, and even substances accidentally present in the soil, as in the case cited by Mr. Doten, add another factor to the complex interplay or corrosive and inhibitive forces affecting the durability of cast iron.

Apparently insignificant factors may produce far-reaching effects. Mr. F. N. Speller,* in studying the corrosion of iron, took two clean iron rods connected by copper wires to a volt meter, and immersed them in salt water. A slight difference of potential was usually shown. If one of the rods was exposed to the air for a few minutes and then replaced in the water, a strong deflection, amounting to 50 milli volts was indicated with the electrode which was aerated as a cathode. On removing the other rod and replacing it after an equal or slightly longer exposure, the current was reversed.

After a prolonged study of the subject Mr. Speller concluded that "the rate of corrosion evidently depends more on the oxygen available than any other factor". Whether the corrosion proceeds uniformly or is concentrated in pits, seemed to him to depend chiefly on the irregular distribution of foreign impurities in contact with the surface, an idea according well with Mr. Doten's theory.

A statement of the writer on page 812 is open to misinterpretation. In the clause reading "The salt of the ocean is particularly destructive to cast iron", it was not intended to imply that wrought iron or steel is less seriously affected than cast iron. The writer is absolutely in accord with Mr. Doten's views on the superior resisting power of cast iron under such conditions, and has so stated on page 853. He is glad to be corroborated so strongly in this view by one having such extended experience in this matter.

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* *Journal*, Engrs. Soc. of Pennsylvania, Vol. 1, p. 243.

Mr. Pugh.

So far as concerns the main conclusions of the paper, the writer sees no reason to change them. The discussion has been of great aid in considering the merits of different remedies, and may be summarized as follows:

A.—Increasing the Skin Resistance of Cast Iron.—This deserves investigation, but the outlook is not particularly promising.

B.—Utilizing the Protective Influence of Alkalis by Surrounding the Pipe with Lime or Cement Where Such is Practicable.—This method was advised in the Joint Report for the protection of the Coolgardie, Australia, main which passed through corrosive soils, and is to be used in connection with the new Birkenhead, England, supply where the main is laid in salt marsh. It was adopted with success in Boston Harbor. The discussion would seem to bear out this method as having a wide field of usefulness.

C.—Exclusion of Acids, Salt, or Air.—Under this caption would come the device of raising the pipe above the surface. The discussion brought out a successful instance of this in a pipe laid in 1862, and still in service. The double dipping of pipe, as outlined in the Joint Report, is also deserving of careful consideration. Wrapping the pipe is also shown to be capable of wide application.

D.—Galvanizing the Cast-Iron Pipe, Thus Protecting It at the Expense of the Zinc.—What little was brought out under this head in the discussion tends to point to it as being desirable under certain conditions. Strong evidence is furnished by the example mentioned by the writer of its protective effect on a sewer outfall.

The engineer should be sufficiently familiar with conditions under which cast-iron pipe is subject to rapid deterioration to advise when it may be desirable to spend even relatively large sums for external protective measures, as well as to appreciate when the cost of such protection would be incommensurate with the results obtained.

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Paper No. 1326

THE CLARIFICATION OF SEWAGE BY FINE SCREENS*

By Kenneth Allen, M. Am. Soc. C. E.

With Discussion by Messrs. G. Bertram de B. Kershaw, W. L. Stevenson, George C. Whipple, C. A. Jennings, G. Franze and Hermann Schaefer, George W. Fuller, J. X. Cohen, F. T. Robson, Alexander Potter, Charles E. Gregory, Rudolph Hering, J. H. Granbery, Langdon Pearse, L. C. Whittemore, Samuel A. Greeley, E. Kuichling, George T. Hammond, J. C. Riedel, David T. Pitkethly, William L. D'Olier, John H. Gregory, George A. Soper, and Kenneth Allen.

Synopsis.

The object in presenting this paper is to bring before members of the Society who are interested in sewage disposal several devices for fine screening that have been installed within the last few years, with such data regarding their merits and defects as the writer has been able to gather from various inspections and by correspondence.

A secondary object has been to indicate the relative position of fine screening as a method of sewage treatment with respect to tank treatment, in the belief that, although the field for fine screening is distinctly limited, its possibilities are not yet fully realized.

No attempt has been made to give other than general or essential details regarding the types of screens discussed.

By fine screens is meant those used in partly removing the smaller organic particles of suspended matter which cause a rapid decrease of the dissolved oxygen of the streams into which they are discharged;

* Presented at the meeting of October 21st, 1914.

and, for the present purpose, such screens are assumed to have a maximum free opening of not more than ⅝ in., or 15 mm.

Five types are considered. Several screens, otherwise excellent, but which are cleaned below the surface, are not considered, as this method of cleaning is believed to be fundamentally bad. All the types discussed are in successful operation in Germany, and there are a limited number in England and the United States. These types are:

1.—The Band Screen, exemplified by that at Hamburg, designed by Brunotte and manufactured by the Buckau Machinery Company, of Magdeburg, Germany, and the Carshalton Screen at Sutton, manufactured by John Smith and Company, of Carshalton, England.

2.—The Wing Screen, of Uhlfelder and Schneppendahl, in use at Frankfort, Germany, and manufactured by J. S. Fries Sohn, of that city.

3.—The Shovel-Vane Screen, in use at Strassburg, Germany, and made by the Geiger Machine Works, of Karlsruhe.

4.—The Drum Screen, designed by G. Windschild, of Dresden-Cossebaude, in use at Trier; and that designed by O. M. Weand, of Reading, Pa., and in use at Brockton, Mass.

5.—The Riensch-Wurl Screen, used at Dresden, and manufactured by Wilhelm Wurl, of Berlin-Weissensee.

To furnish the best results, a screen should be efficient in the removal of material, reliable under conditions to be met in operation, free from nuisance, accessible for repairs, and inexpensive in first cost and operation. In general, the screens described in this paper meet these conditions quite well. Naturally, other things being equal, the finer the screen the more efficient it is, but collected data are so discordant that the desirability of standardizing the methods of obtaining and recording results is made very evident.

It is suggested that special attention be given to the mode of sampling, that samples be weighted in accordance with the flow, and that the amount of preliminary cleaning by coarse screens and grit chambers be always noted.

Except the Hamburg screen, which is somewhat coarser than the others, the fine screens now in regular operation in Germany will ordinarily remove more than 30% of the suspended solids, two-thirds

or more of which consist of organic matter. This fact, which is not yet generally recognized in America, places fine screening in competition with tank treatment, although, usually, its efficiency is somewhat less.

In general, fine screening should be considered:

1. Where the removal of all but the very fine solids will furnish a satisfactory effluent;
2. Where land values are so high as to render sedimentation installations costly;
3. Where excavation for tanks would be costly;
4. Where the disposal of sewage sludge would be expensive or cause a nuisance;
5. Where the recovery of grease or fertilizing value is important.

The Removal of Suspended Solids from Sewage.

The discharge of crude sewage into streams is objectionable, primarily, on account of the grosser solids held in suspension or floating on the surface. These are offensive to the eye, and cause foul deposits on river and harbor bottoms, which produce objectionable odors and, by depleting the water of its dissolved oxygen, reduce its capacity to digest the impurities which remain.

Although these dissolved impurities as well as the pathogenic bacteria are left, by far the greater part of the nuisances from sewage effluents are due to the suspended solids. Moreover, where finishing processes are required for the oxidation of the impurities held in solution, or for sterilization, it has been found desirable to remove a part of the solids as a preliminary measure, in order to facilitate these more refined processes. Hence methods of eliminating solids from sewage are of primary importance and are being adopted quite generally by progressive cities.

The two practicable and standard methods of accomplishing this are by allowing the solids to deposit by settling in tanks, from which they are removed at intervals, and by intercepting them by screens. Ordinary sewage in American cities contains about 700 parts per million of total solids, of which 250 parts are in suspension. Of these, some 50 parts per million are so finely divided as to be either colloidal or approaching this condition, and are incapable of removal by ordinary methods. The percentage of suspended matter removable by tank

treatment depends on the parts per million in the crude sewage, the length of retention in the tank, the velocity through the tank, and its design; but, with ordinary sewage and with good design and operation of the tank, one may expect a removal:

With plain sedimentation, of about:

65% of the suspended solids, and

30% of the total organic matter.

With chemical precipitation, of about:

80% of the suspended solids, and

50% of the total organic matter.

It is commonly stated that from 15 to 20% of the suspended matter may be removed by screening, under favorable conditions. It is believed by the writer that the time has come when these figures should be revised; that modern improvements in screen design have reached the point when screening may fairly be compared with tank treatment in the percentage of suspended solids removed; in addition to which it is generally the case that an analysis will indicate a higher percentage of organic matter—or that most objectionable to discharge into streams—per pound of dried solids in screenings than in settled sludge.

Types of Screens.

In general, the screens in use consist either of parallel bars or wires, perforated plates, or woven rods or wires.

Coarse Screens.—Screens of this type are in common use before pumps to protect the valves and prevent the obstruction of water passages. In their simplest form, they consist of a series of vertical or sloping bars, usually from 1 to 4 in. apart, and are cleaned by hand (Figs. 1, 2, and 3). In some large installations the bars form the sides of a cage with a perforated bottom, which is raised by power to an operating floor where it is cleaned by hand. This form is used at the Ward Street pumping station and the several outfall works at Boston, Mass. (Fig. 4). In England, the fixed screen of sloping bars is frequently cleaned automatically by rakes which engage between the bars near the floor of the screen chamber, draw the screenings to the top of the screen, and deposit them on a band conveyor. Still another type of coarse screen is in the form of a basket of interlaced rods through which the sewage passes downward. This screen is raised for cleaning. A preferable

form is a rotary screen which is cleaned above water, such as that at the Crossness Outfall Works at London (Fig. 5). In this example grit also is removed by buckets attached to the screen. The spaces in this screen are ¾-in.

According to E. Kuichling, M. Am. Soc. C. E., coarse screens generally remove from 3 to 10 cu. ft. of material per million gallons of sewage, equivalent to about 104 lb. of dried matter or 12½ parts per million of the suspended solids.

Fine Screens.—These screens are in use to a limited extent, both in America and England, but their most marked development has been brought about in Germany. This is perhaps chiefly due to the fact that, though there are many large cities in that country, they are mostly on large streams, such as the Elbe and Rhine, so that, in many cases since 1903, the Government has sanctioned processes which merely retain solids more than 3 mm. in diameter. In such cases, screens may be substituted for the more expensive tank installations. The result has been an introduction of several types of fine screens in many towns during the past few years.

It is not easy to draw a definite line between fine and coarse screens, but, for the present discussion, we may consider those having a clear opening or mesh not greater than 0.6 in., or 15 mm. as "fine".

From experiments by Monti at Berlin, it was concluded*

"that most of the suspended matter in sewage is usually in a very fine state of division, or capable of passing through a sieve having meshes 0.02 inch square; that most of the solid fecal matter found in the liquid can be arrested on a sieve having meshes 0.25 inch square; and that a sieve with meshes 0.10 inch square is probably fine enough to extract from the sewage all suspended matter of appreciable magnitude, and especially such as is likely to become offensive [under conditions proposed for Rochester]. * * * It is believed, * * *, that ample evidence has been submitted to show that a surprisingly large quantity of offensive organic matter can be extracted from sewage by properly designed screening appliances, and that the screened liquid may then be admitted safely for final disposal into bodies of water which would soon become repulsive without such screening."

Function of Fine Screens.

The function of fine screens is quite different from that of coarse ones, so that we may be justified in considering them in a class by

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* Sewage Disposal System. Rochester, Kuichling, 1913.

FIG. 1.—COARSE SCREEN AT PROVIDENCE, R. I.

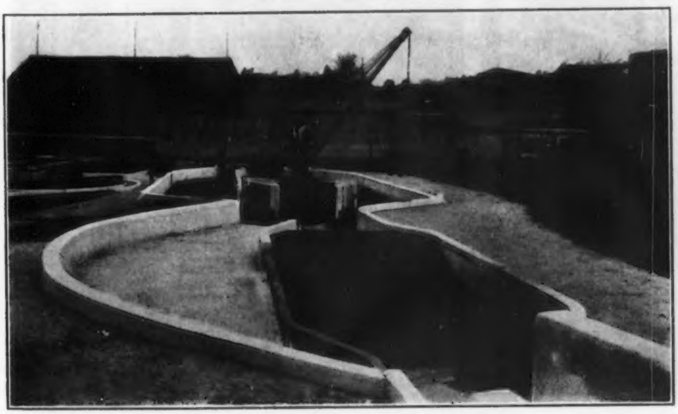


FIG. 2.—COARSE SCREEN AT ESSEN, N. W.

FIG. 3.—COARSE SCREENS AT COLOGNE.

FIG. 4.—CAGE SCREEN AT BOSTON, MASS.

FIG. 5.—COARSE BAND SCREEN AT CROSSNESS, LONDON, ENGLAND.

FIG. 6.—BAND SCREEN AT HAMBURG.

themselves, presenting problems of design and operation that have no similarity to the design or operation of coarse screens. The use of fine screens may be advisable for any of the following purposes:

1. The removal of matter that otherwise would cause the pollution of the stream receiving the effluent;
2. The removal of solids that otherwise would cause the frequent subsequent clogging of sprinkler nozzles or filters; and
3. The removal of material likely to promote the formation of scum in subsequent tank treatment.

In Germany their use has been justified mainly for the first reason, but it is recognized that fine screening before applying sewage to sprinkling or sand filters may cause a marked reduction in operating expenses. In Baltimore fine screens have been placed between the tanks and sprinkling filters, with the result that whereas, previously, "two men, eight hours per day, were always required and sometimes more for cleaning nozzles for between six and nine acres of filter area, with the screen in operation two men are required to spend about two hours a day each for the same number of nozzles."*

According to the Massachusetts State Board of Health: "When practically all the suspended matters were removed from the sewage the possible rate of filtration [to sprinklers] could be doubled."†

On the other hand, their use for this purpose has been discontinued at Birmingham, England, and at Atlanta, Ga., as unnecessary after efficient tank treatment. On the whole, it is questionable whether their expense is often justifiable for this purpose. Mr. Francis E. Daniels‡ states that "the use of fine screens will entirely prevent the formation of tank scum." If this is substantiated by further experience, it will be an argument in their favor in connection with those sedimentation plants where the formation of scum has been found to be a nuisance.

Conditions to be Satisfied.—The conditions to be satisfied in a fine screen may be summarized as follows:

a. It must remove a sufficient quantity of the suspended solids and total organic matter in regular operation to satisfy the

* Letter from H. C. McRae, Assoc. M. Am. Soc. C. E., Division Engineer. Baltimore Sewerage Commission, February 24th, 1914.

† *Surveyor and Municipal and County Engineer*, March 6th, 1914.

‡ Director, Water and Sewerage Inspection, State Board of Health, New Jersey, *Municipal Journal*, January 15th. 1914.

needs of the situation. In other words, its efficiency must be adequate.

b. In operation, it must be reliable; and, in capacity, it must be ample, under the conditions of sewage flow and surface elevation which may obtain.

c. Its operation must be free from nuisance and unsanitary conditions.

d. All parts must be accessible for inspection, repair, and renewal.

e. The cost of installation must be low.

f. The costs of operation, renewals, and repairs must be low.

These conditions will be considered subsequently, in their relation to different screens, under the heading, "Comparison of Screens."

Types of Fine Screens.

The fine screens in successful operation may be classified under five distinct types, all of which are found in Germany. In all of them the screenings are raised gently above the surface before being removed, instead of being raked off while submerged, as is done with some of the coarser screens in England, and at Paris, Cologne, and elsewhere.

These types are as follows, in the order of the usual size of their opening (Fig. 7)*:

1. The band screen (*Abfischgitter*), consisting of an endless flexible band, either of wire mesh or links, which passes over upper and lower rollers.

2. The wing screen (*Klärrechen*), formed of vanes, as in a paddle-wheel, which are composed of radial bars at uniform distances apart.

3. The shovel-vane screen (*Siebschaufelrad*), similar to the wing screen, but with semicircular wings and a different method of removing the screenings.

4. The drum screen (*Siebtrommel*), consisting of a cylinder or cone of perforated plates or wire mesh, which rotates on a horizontal axis.

5. The Riensch-Wurl screen (*Separatorscheib*), which consists of a perforated disk surmounted by a truncated cone, also perforated. The disk is mounted on an inclined shaft.

* Except that the wire mesh used in any type may be very fine.

The fourth and fifth may have openings of 1 mm. or less. Descriptions of each, with typical examples, are given.

Band Screens.

Band screens may be broadly divided into two classes: (*a*) those composed of bars or links; and (*b*) those composed of a fine mesh.

a.—Hamburg.—The Hamburg screen (Fig. 6) at the main outfall works on the north side of the Elbe, is composed of a series of parallel links, 14.2 in. (36 cm.) long, 0.2 in. (0.5 cm.) thick, and

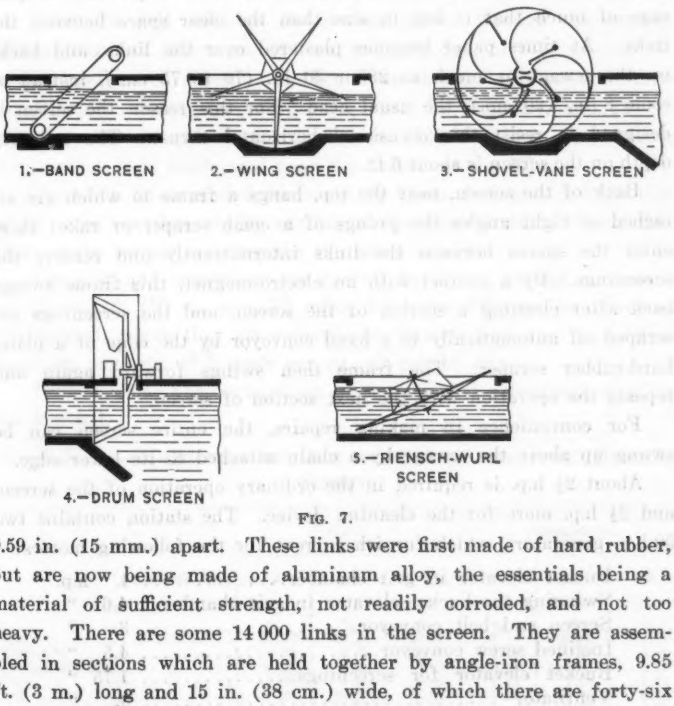


Fig. 7.

0.59 in. (15 mm.) apart. These links were first made of hard rubber, but are now being made of aluminum alloy, the essentials being a material of sufficient strength, not readily corroded, and not too heavy. There are some 14 000 links in the screen. They are assembled in sections which are held together by angle-iron frames, 9.85 ft. (3 m.) long and 15 in. (38 cm.) wide, of which there are forty-six in each of the two screens installed. The bars are readily removable from the frames for purposes of repair.

Each of the two screens at this station is about 33 ft. (10 m.) long, 10.8 ft. (3.3 m.) wide, and 19 ft. (5.8 m.) high, over all. Their inclination to the horizontal is about 63 degrees. This great

length has been made necessary by the tidal range of 6.2 ft. to which the screens are subjected. They are supported and operated by wheels resting on rails, on each side, which pass over sprocket wheels mounted on a shaft at each end, the upper shaft being connected by gearing to an electric motor.

As the assembled frames of links forming the screen band move upward with a velocity of from 1 to 2 in. per sec., they carry with them, above the surface of the sewage, the material intercepted. As larger particles of material are caught, they serve to prevent the passage of much that is less in size than the clear space between the links. At times paper becomes plastered over the links, and backs up the sewage as much as 28 or 30 in. (70 to 75 cm.), instead of some 8 in. (20 cm.), the usual head. For this reason the screen is designed to resist this occasional horizontal thrust. The ordinary depth on the screen is about 6 ft.

Back of the screen, near the top, hangs a frame to which are attached at right angles the prongs of a comb scraper or rake; these enter the spaces between the links intermittently and remove the screenings. By a contact with an electro-magnet, this frame swings back after cleaning a section of the screen, and the screenings are scraped off automatically to a band conveyor by the edge of a plain, hard-rubber scraper. The frame then swings forward again and repeats the operation with the next section of screen.

For convenience in making repairs, the entire screen can be swung up above the sewage by a chain attached to its lower edge.

About 2½ h.p. is required in the ordinary operation of the screen, and 2½ h.p. more for the cleaning device. The station contains two 30-h.p. generators, which furnish current for the following motors:

Bucket elevator in grit chamber.............	4.	h.p.
Swinging the bucket elevator in grit chamber..	4.6	"
Screen and belt conveyor....................	3.	"
Inclined screw conveyor......................	4.5	"
Bucket elevator for screenings..............	1.75	"
Ventilator	3.	"

In addition, current is furnished for 4 arc lamps, 74 incandescent lamps, and for feeding a storage battery used for the illumination of boats in the main outfall sewer.*

* "Die Kanalisationsanlagen der Freien und Hansestadt Hamburg," Curt Merckel, Hamburg, 1910.

The population now tributary to the works is about 1 000 000, and the sewage flow is 53 000 000 gal.* (190 000 cu. m.) per day. This varies from 140 cu. ft. per sec. (4 000 liters per sec.) during dry weather to 700 cu. ft. per sec. (20 000 liters per sec.) during storms.

On entering the station the sewage first flows through a grit chamber, 54 ft. long, 30 ft. wide, and about 6½ ft. deep below the normal surface of the sewage. About 9 cu. yd. (7 cu. m.) of sand and other heavy material are removed per day here, or 0.17 cu. yd. per million gallons.

From the grit chamber the sewage passes through the screens. These take out 18 cu. yd. (14 cu. m.) per day, or 0.34 cu. yd. per million gallons of sewage. It then flows through tide-gates to three riveted-steel outlet pipes, 6.56 ft. (2 m.) in diameter, and 230 ft. (70 m.), 328 ft. (100 m.), and 436 ft. (133 m.) long, respectively, from which it is discharged near the bottom of the Elbe.

Operation is suspended for about 2½ hours during each high tide, or 5 hours per day, as the surface of the river is then higher than that of the sewage, causing the tide-gates to close.

The grit and screenings are carried by scows, having a capacity of 90 cu. yd. (70 cu. m.), 19 miles down the Elbe to the Island of Waltershof, where they are removed by farmers and used to fertilize land, on payment of 7 cents per cu. yd. by the city.

Table 1 shows the composition of the grit and screenings, as given by Merckel.

TABLE 1.—Composition of Grit and Screenings from Hamburg Screens.

	Grit.			Screenings.		
Analysis No.	1	2	3	1	2	3
	%	%	%	%	%	%
Water	65.35	55.90	22.87	83	92.62	85.24
Nitrogen	0.52	0.27	0.21	0.78	0.27	0.02
Phosphoric acid	0.52	0.29	0.40	0.42	0.18	0.12
Fats	1.83	1.28	0.44	7.88	1.68	5.11
Ash	20.28	37.15	71.28	1.33	1.10	5.16
Of which, Sand	......	35.33	67.86	......	0.32	2.88

From analyses of five samples of mixed detritus of varying character from the grit chamber and screens, Dr. Gillmeister, City Chem-

* 190 000 cu. m. from 950 000 population in 1910, "Die Städtische Abwässerbeseitigung in Deutschland," Salomon, 1910, Vol. III, p. 230.

ist, has estimated the value of the contained nitrogen and phosphoric acid as fertilizer, as given in Table 2.

TABLE 2.—VALUE OF CONTAINED NITROGEN AND PHOSPHORIC ACID AS FERTILIZER.

Sample No.	Percentage of moisture.	Percentage of N in dried material	Percentage of P_2O_5 in dried material	Percentage of grease in dried material	Value of Dried Material. In dollars per ton.	Value of Dried Material. In Marks per 100 kg.	Description of material.
I	92.62	3.69	2.34	22.85	$16.87	7.81	Loose; lumpy; somewhat plastic, sticky; fecal odor.
II	22.87	0.27	0.52	0.57	1.62	0.75	Blackish; sandy; mixed with particles of wood and coal.
III	55.90	0.61	0.67	2.91	3.09	1.43	Blackish; sandy; fine; odor slightly fecal.
IV	85.24	0.10	0.72	34.63	1.16	0.54	Loose and fibrous; slightly sandy; slight odor.
V	59.79	1.06	0.83	1.98	5.03	2.33	Rather loose, sandy; dusty.

When mixed with four parts of street sweepings, the grit and screenings are disposed of by incineration.

The first cost of this plant is given by Merckel as follows:

Foundations, etc.	$174 000
Screen house	10 500
Machinery	33 300
Storage house at dock	2 900
Outlet pipes	53 500
Total	$274 200

Up to the present time, it has cost about $300 000.

These screens were designed by Brunotte, and are manufactured by the Buckau Machine Fabrik, of Magdeburg-Buckau.

Four similar screens, although only about 6 ft. (1.8 m.) wide and with 0.4-in. (10-mm.) spaces between the links, have been installed more recently on the south side of the Elbe. These are operated by Diesel motors, and require only about 1 h.p. each. The present population tributary to this plant is about 10 000, increased during the day by 20 000 workmen who live on the north side of the river.

The grit chamber and screens together remove about 11.7 cu. yd. (9 cu. m.) of material daily, one-third of which is organic, indicating a higher efficiency than in the older works. This is probably due in part to a lower velocity (4 in. = 10 cm. per sec.) in the grit chamber than at the North station (19.7 in. = 50 cm. per sec.).

The South Hamburg station is designed to care for three times the present flow.

b.—Göttingen.—Other band screens, in which a wire mesh takes the place of links, are used in a number of places. There are two such at Göttingen, installed in 1903, which will serve as an illustration.

Here the bands are 55¾ ft. (17 m.) long, and are set at an angle of 55° with the horizon. They are composed of 0.06-in. (1½-mm.) copper wire, twisted spirally, and woven with a 0.4-in. (10-mm.) mesh, which passes over two drums, 20 in. (50 cm.) in diameter. The upper drum is belt-driven from an undershot wheel, and the lower one is placed in a sump, 28 in. (70 cm.) deep. Brass angles are attached to the screen, 3.3 ft. (1 m.) apart, to catch matter tending to roll back into the sewage. The screen is cleaned by a brush and water jets.

The material is delivered to a tip-wagon having a double interior screen on one side for draining off the moisture, and is then composted with peat dust and sweepings for a fertilizer.

In operation, the screen moves with a velocity of about 8 ft. per min. (4 cm. per sec.).

The lower end of the screen can be raised above the surface, as at Hamburg, for inspection and cleaning. It is also lifted at night when a 2-in. (5-cm.) bar screen above is lowered to take its place.

As the band stretches, it has been found necessary to tighten it from time to time. This results in a slight narrowing, so that wooden fillers are inserted to prevent leakage between the screen and the sides of the channel.

Every 5 or 6 weeks the screens require a thorough cleaning, and the sediment that has collected in the channels is flushed out.

As Göttingen is sewered by the separate system, grit chambers have not been found necessary. In 1907, when the population was 35 000, 38% of the houses were supplied with water-closets. The

screenings amounted to 9.56 cu. yd. (7.3 cu. m.)* per 1 000 population.† The screenings per 1 000 population per day were, therefore, $\frac{9.56}{365} = 0.026$ cu. yd. The sewage flow was then about 2 600 000 gal. per day,‡ so that about 0.35 cu. yd. was removed per million gallons. The suspended matter is said to have been reduced 90 per cent.§ The total cost of the works was $18 500 (60 700 Marks), of which the screens and machinery cost $3 600 (15 000 Marks). The screens were installed by City Engineer Hertzberg.

There are two other fine band screens to be mentioned.

The Carshalton Screen.—This screen (Fig. 8) is similar in principle to those at Göttingen, but the slope is not so great—usually 30° with the horizon—and it is "made of perforated steel strips, twisted steel wires and pins, similar to a coal picking belt. The holes are usually ⅜ in. in diameter." It is operated by an undershot wheel placed behind the screen.

These screens are installed at Birmingham, Chester, Croydon, Wrexham, Ipswich, and about a dozen more English towns, generally between tanks and oxidizing beds. They are manufactured by John Smith and Company, of Carshalton. According to the manufacturers, they will remove about 2½ cu. yd. of material per million U. S. gallons of sewage, varying, of course, with the character of the latter. A screen with a 6-ft. width will care for about 2 750 000 U. S. gal. in 24 hours. At Sutton, one of these screens, with a ¼-in. mesh, removes 1 200 lb. of screenings per million gallons.||

The Jennings Screen.—This screen (Fig. 9) consists of a fine wire-mesh band, and has been in operation since July, 1913, at the Chicago Stock Yards, with a flow of from 1 250 000 to 1 500 000 gal. per day. Although this may be said to be still in the experimental stage, a brief reference to it will not be out of place. The wires are of Monel metal, with 40 meshes per inch. The screen is composed of separate sections of removable frames, for convenience in

* In Frühling's "Entwässerung der Städte", p. 522, he gives for April, 1907, a recovery of 23 100 kg., equivalent to 281 050 kg. per annum, from a population of 35 000, or 8 030 kg. = 7.3 cu. m. per 1 000 (instead of 7.9 cu. m. per 100, as then stated by him).

† Based on the weight of screenings, assuming their specific gravity to be 1.1.

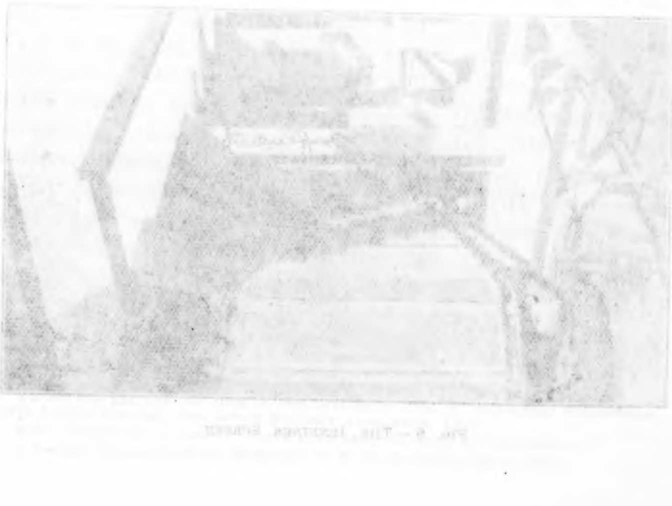
‡ 2 200 000 gal. per day from 30 000 persons in 1905, E. Kuichling, *Proceedings*, Fourth Annual Meeting, New Jersey Sanitary Association, 1908.

§ Mr. Kuichling has stated: "There is probably an error in this estimate."

|| Sewage Disposal System, Rochester, N. Y., E. Kuichling, April, 1913.

FIG. 8.—CARSHALTON SCREEN, AT WREXHAM, ENGLAND.

FIG. 9.—THE JENNINGS SCREEN.



repairs and renewals. The screenings are blown off by air compressed to 1½ lb. per sq. in. at the slotted jet used for cleaning. A 10-h.p. motor is used for operating the screen and blower, although less power is required after starting. The entire cost for air and power amounts to from 12 to 15 cents per hour. The plant is looked over once or twice a day by one of the regular employees so that there is no extra charge for attendance.

According to C. A. Jennings, Assoc. Am. Soc. C. E., the designer, "with an influent running about 700 p.p.m. suspended matter, the screen removed about 63% * * *. The screenings contained an average of about 79% moisture."

In one test of 6 hours' duration, the wet screenings amounted to 6 186 lb. per million gallons. Another test of 5 hours gave 4 828 lb. per million gallons, the suspended matter in this case being 20% lower than in the former.

Features of this screen are the fineness of the mesh and its construction in sections, as at Hamburg.

Wing Screens.

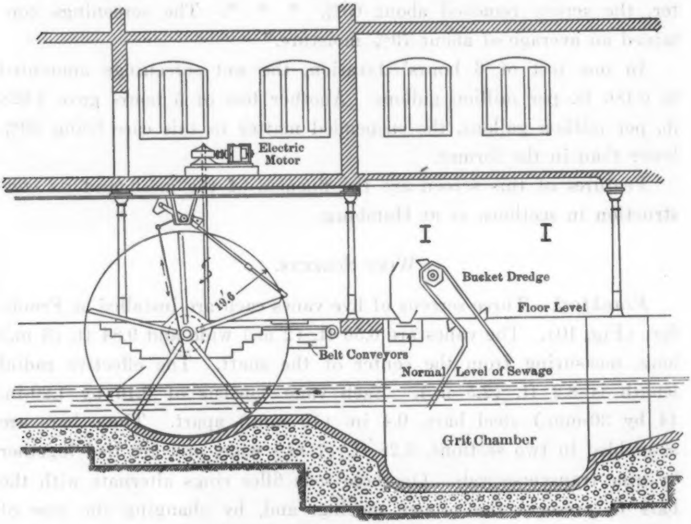
Frankfort.—Three screens of five vanes each are installed at Frankfort (Fig. 10). The vanes are 6.56 ft. (2 m.) wide and 9.84 ft. (3 m.) long, measuring from the center of the shaft. The effective radial length is 8.2 ft. (2.5 m.). Each vane consists of 0.16 by 1.20-in. (4 by 30-mm.) steel bars, 0.4 in. (10 mm.) apart. These bars are assembled in two sections, 3.28 ft. (1 m.) wide, and are tied together by nine transverse rods. On these rods filler rings alternate with the bars to preserve the proper spacing, and, by changing the size of these rings, the spacing may be altered. The ratio of the free opening to the entire vane is 1 : 2.4. To avoid undue friction in the passage of the sewage through the screen, these bars are given a less depth between the holes for the tie-rods.

The rectangular channel is depressed at the screen, so that during a revolution one vane will occupy the entire cross-section of the stream at all times and prevent the by-passing of an appreciable quantity of sewage.

As the lowest vane, moving against the current, rises slowly, a hard-rubber scraper, hung by a long arm at each end, is forced by the motion of the vane to travel from its inner portion near the

shaft to the extremity of the bars. This scraper is, or may be, followed by a brush for the more thorough cleansing of the bars. By the time the vane is in a horizontal position, the screenings reach its outer edge and drop on a horizontal steel plate. This receiving plate then recedes from the screen, and by this motion the detritus is scraped off by a stationary vertical plate and falls on a horizontal belt conveyor. This carries it laterally to a tip-car for removal.

Each screen is operated by a motor placed above it, the speed of which is adjusted to the work to be performed. The motion of the



WING SCREEN AT FRANKFORT

FIG. 10.

receiving plate is transmitted by mechanism from the scraping apparatus, so that it occurs at precisely the time required. The belt conveyor has a velocity of 3¼ ft. (1 m.) per sec.

The sewerage of Frankfort is on the combined system, and the population is about 420 000. The volume varies from 21 000 000 to 26 000 000 gal. (80 000 to 100 000 cu. m.) per day. The daily per capita flow, therefore, is about 56 gal. (213 liters).

After passing a grit chamber, where 20 tons of detritus are removed daily, or, say, 0.8 ton per million gallons, the sewage flows

through the screens, two of which are generally in use. They remove automatically 17 cu. yd. of screenings per day, or 0.7 cu. yd. per million gallons of sewage, composed largely of feces and paper.

Measurements made about 7 years ago indicate that 16% of the suspended matter is removed in the grit chamber and 10% more by the screens, making 26% in all. At that time the water supply was 47 gal. and the sewage flow 39½ gal. per capita. Table 3 shows the quantities of material removed by the grit chamber and the screens, as given by Uhlfelder and Tillmans.*

TABLE 3.—Material Removed by Grit Chamber and by Screens, in Parts per Million.

	In Raw Sewage:									Intercepted by:								
	Dissolved.			Suspended.			Total.			Grit Chamber.			Screens.			Both.		
	Total.	Organic.	Mineral.	Total.	Organic.	Mineral	Total.	Organic.	Mineral.	Total.	Organic.	Mineral.	Total.	Organic.	Mineral	Total.	Organic.	Mineral.
Day sewage, 8 A. M. to 2 A. M..	783	230	553	485	285	200	1 268	515	753	65	39	26	28	24	4	93	63	30
Night sewage, 2 A. M. to 8 A. M..	615	141	474	191	110	81	806	251	555	61	32	29	5	5	0	66	37	29
24-hour sewage..	741	208	533	411	241	170	1 152	449	703	64	37	27	22	19	3	86	56	30

Of the whole quantity, about three-quarters were removed by the grit chamber and one-quarter by the screens. The grit was about half organic and half mineral, and the screenings were from 86 to 100% organic. These relations, of course, depend on the design and operation of both grit chamber and screens; but, at Frankfort, although the grit chamber removes three times as much material as the screens, the latter remove half the organic matter taken out. To operate one of these screens requires 2.3 h.p.

During eight years of service, the repairs and renewals are said to have been insignificant. The annual cost of operation is given by the late City Engineer Koelle as 11 cents per capita. It is otherwise stated as 18 cents per cu. yd. (0.75 Mark per cu. m.) of screenings, or about one-sixth of the cost when removed by hand.

* "Die Frankfurter Kläranlage," *Wasser und Abwasser*, Vol. 1, No. 7, 1909. *Mitteilungen a. d. Königl, Prüf. für Wasserversorgung und Abwasserbeseitigung*, Vol. 10, 1908.

Elberfeld.—At Elberfeld there are two screens (Fig. 11), similar to those at Frankfort, taking the sewage from a population of about 340 000 persons in Elberfeld and Barmen. The sewers are mostly on the separate system, and the volume of sewage is 16 000 000 gal. (60 000 cu. m.) daily. It contains 500 parts per million of suspended material.

The sewage is quite fresh when received at the works. It first enters a grit chamber where two dredges take out the coarse material—largely kitchen refuse, with a little sand. It then passes the screens. These rotate once in 3½ min. and, with the grit chamber, remove 18.2 cu. yd. (14 cu. m.) of material per day, or 1.15 cu. yd. per million gallons. As at Frankfort, these appeared to consist chiefly of feces and paper. They are placed in unprotected dumps for drying, and are naturally very offensive in warm weather.

The screens are cleaned about once a week, while in operation, by raking out the bars by hand. This takes one man about ½ hour for the two screens.

The power required for operating a grit elevator, a sludge pump, and the screens is usually 5 h.p., but, for short periods, may be as much as 10 h.p.

The Elberfeld screens are placed before settling tanks, the excuse for this preparatory treatment being the removal of a substantial part of the solids in a form containing relatively little moisture and, therefore, readily handled.

At the works of J. S. Fries Sohn, at Frankfort, where the Frankfort and Elberfeld screens were manufactured, the writer was told that one such screen would ordinarily remove 16 cu. yd. (12 cu. m.) of material from 8 000 000 gal. of sewage (30 000 cu. m.) per day, or 2.0 cu. yd. per million gallons, and that for operating the screens 3 h.p., or, with auxiliary machinery, 5 h.p., would be required.

Bradford, England.—Two screens of the Frankfort type were installed at Bradford a few years ago, but differ from the former in some details (Fig. 12). Each is 6 ft. 2½ in. wide, with length of vane of 4 ft. 11½ in. and a capacity of 3 500 000 gal. per day. The bars are about ¼ by 1 in. in section, and ½ in. apart. There are no cross-rods except near the extremity of the vane. A chief function of these screens is to intercept wool fiber, which occurs in considerable quantity in the Bradford sewage. For this purpose the wing

Fig. 11.—Wing Screen at Elberfeld.

Fig. 12.—Wing Screen at Bradford, England.

[illegible]


RIDGE

[illegible]

screen has proved well adapted. In operation, the screen comes to rest when a pair of vanes reaches the horizontal position, and a rake with teeth interlocking with the screen bars is introduced and withdrawn mechanically, removing the material intercepted. A few fibers clinging to the cross-rod occasionally require removal by hand. For each one-sixth of a revolution 33 sec. are required, and an equal interval for cleaning.

Wiesbaden.—At Wiesbaden a screen of this type, with six vanes, was put in by Schneppendahl in 1899. The vanes are 6.56 ft. (2 m.) wide, and the bars are 0.6 in. (15 cm.) apart. It is operated by hand, day and night. The sewage amounts to about 3 000 000 gal. (11 000 cu. m.) daily from 100 000 persons, is domestic in character, and contains much soap and fats. It first passes a fixed screen with 1.6-in. (40-mm.) spaces. The fine screen removes 3.4 cu. yd. (2.6 cu. m.) of screenings, largely feces, or 1.1 cu. yd. per million gallons.

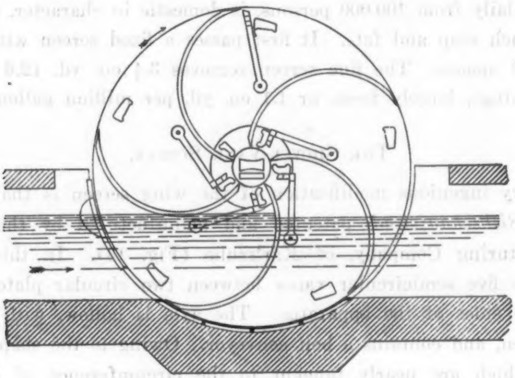
The Shovel-Vane Screen.

A very ingenious modification of the wing screen is that known as the *Siebschaufelrad,* patented and manufactured by the Geiger Manufacturing Company, of Karlsruhe (Fig. 13). In this screen there are five semicircular vanes between two circular plates which form the sides of the apparatus. The axle is hollow, with the top third open, and contains a belt conveyor. Owing to the shape of the vanes, which are nearly tangent to the circumference of the side plates on their outer extremities and to the radius at their inner edges, the sediment and floating matter are scooped up gently above the surface of the sewage, with little loss or disintegration, and finally drop by gravity from the screen to the belt conveyor. Such material as adheres to the vanes is brushed off mechanically when above the axle by a brush on the end of an arm swinging from a center near the middle of the adjacent vane. This brush is of piassava fiber, which is well adapted to the purpose. By a mechanical device it comes into contact with the vane only while swinging in an inward direction over the part of the vane above the axle. At other times, it rests near the axle, and, therefore, is ordinarily above water. In the operation there is little tendency to crush the screenings or press them through the screen itself.

The screen vanes are of pressed wires of V-section, and are 1, 2, or 3 mm. apart, and held in place by a central rib and by round transverse tie-rods.

Power is transmitted to the wheel by gearing. The motion is slow and uniform. The moving parts are open to inspection and repair above the level of the sewage.

Paper, which might partly clog the screen and obstruct the flow, is retained by a baffle placed before the screen, but sand or gravel which may pass the grit chamber above is handled and removed with the other screenings. There is a clearance of only 0.04 in. (1 mm.)



GEIGERS SHOVEL-VANE SCREEN

FIG. 13.

between the screen and the curved bottom of the channel, and this space is prevented from clogging by the constant motion of the screen against the current.

Shovel-vane screens are in operation at Strassburg and Gleiwitz, Germany, and at Temesvar, Hungary.

Strassburg.—A population of 160 000 contributes daily 11 900 000 gal. (45 000 cu. m.) of rather weak domestic sewage containing trade wastes from the city and the suburbs, Neudorf, Kronenburg, and Königshofen-Gruneberg. It arrives while quite fresh, and first passes two vertical screens of bars 0.32 in. (8 mm.) apart, which are cleaned by a movable scraper.

Below these bar screens a Geiger screen was put in as an experiment in August, 1908, with 0.10-in. ($2\frac{1}{2}$-mm.) openings and a width of 5.25 ft. (1.6 m.). Having been erected in an old grit chamber, the ratio of submergence to depth of sewage is said to have been unfavorable for the best results, the diameter too small, the axle too low with reference to the surface, and the free opening too small, considering the width of the channel above, which is 9.2 ft. (2.8 m.). Nevertheless, the capacity of the screen, with the sewage backed up 12 in. (0.3 m.), is about 18 000 000 gal. per day (800 liters per sec.). The small quantity of hair and paper which collects on the screen is removed by a jet from a hose operated by a plunger pump for about 15 min. every morning. The piassava brushes last about 2 months, and are readily renewed at a cost of from \$83.50 to \$95.00 (from 350 to 400 Marks).

The quantity of screenings and grit removed daily amounts to $5\frac{1}{4}$ cu. yd. (4 cu. m.), or about 0.034 cu. yd. (0.025 cu. m.) per 1 000 inhabitants tributary; and this material is delivered by conveyors to scows in the river, or, by reversing the motion of the conveyor, to a pit from which it is removed by farmers.

The force at the plant consists of a superintendent, a machinist, and two men who attend to the screen, load and care for the scows, etc.

From 1 600 to 2 000 cu. yd. (1 200 to 1 500 cu. m.) of screenings, coarse and fine, and grit, are removed annually at a total cost of \$3 570 (15 000 Marks), or, say, \$2 per cu. yd. (9 Marks per cu. m.).

The cost of operation of the fine screens, exclusive of labor, is 30 cents (1.25 Marks) per 10-hour day, and the power consumed is 25 kw.

About 1.3 cu. yd. (1 cu. m.) of fine screenings are removed per day from the sewage of 30 000 persons, at a cost of 5.4 cents per cu. yd. (0.30 Mark per cu. m.).

Gleiwitz.—The population of Gleiwitz is about 67 000, and produces a volume of sewage varying from 1 000 000 to 2 600 000 gal. (4 000 to 10 000 cu. m.) per day. Two Geiger screens (Fig. 14), 12.5 ft. (3.8 m.) in diameter and 5.9 ft. (1.8 m.) wide, are installed. The screen vanes are composed of **V**-shaped rods, 0.12 in. (3 mm.) apart. The quantity of material removed per year is 4 700 cu. yd. (3 600 cu. m.), or 0.19 cu. yd. per 1 000 persons daily, at an annual cost of from \$480 to \$710 (2 000 to 3 000 Marks). The cost, therefore, is

probably about 90 cents per million gallons, and the cost of screenings, 12½ cents per cu. yd.*

Although the average quantity of screenings is 12.9 cu. yd. per day, about 21 cu. yd. (16 cu. m.) have been removed in 1 day, and 5 cu. yd. (4 cu. m.) in 2 hours, by the two screens.

The cost of each screen was \$2 800 (11 750 Marks), and that of the entire plant \$6 860 (28 800 Marks).

Two men are required to attend the screens and pumps, one during the day and one at night. The power for operation varies from 1 to 2 h.p.

Analyses of the sewage before and after screening are given in Table 4.

TABLE 4.—ANALYSES OF SEWAGE AT THE GLEIWITZ SCREENING PLANT, IN PARTS PER MILLION.

	Before screening.	After screening.
Reaction	Weakly alkaline.	Weakly alkaline.
Appearance	Brown, strong, with coarse material; turbid, with light settleable particles.	Yellowish brown, turbid, with fine suspended matter.
Suspended solids	2 112.8	785.2
Dried residue after filtration	1 237.2	1 260.4
Loss on evaporation	400.0 = 33.08%	388.8 = 30.85%
Oxygen consumed	105.5	100.6
Ammonia	30.0	30.0
Chlorine	262.2	279.4
Sulphuric acid	Present.	Trace.
Nitrous acid	None.	None.
Nitric acid	None.	None.

The clarifying efficiency, therefore, was $\frac{2\,112.8 - 785.2}{2\,112.8} = 62.9\%$; on the other hand, the solids in solution (probably colloidal matter) increased from 1 237.2 to 1 260.4 parts per million, or about 1.9 per cent.

Temesvar.—The tributary population at this place is about 60 000, and the rate of flow through the screens varies from 3 650 000 to 9 150 000 gal. per day (160 to 400 liters per sec.). Provision is made for a flow of 13 700 000 gal. per day (600 liters per sec.), beyond which the excess is by-passed.

There are two Geiger screens, 11.8 ft. (3.60 m.) in diameter and

* *Gesundheits-Ingenieur*, December 21st, 1912.

FIG. 14.—SHOVEL-VANE SCREEN AT GLEIWITZ.

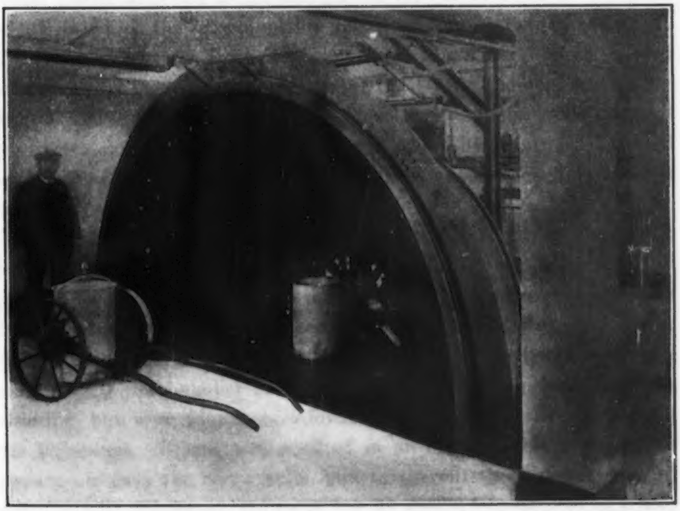


FIG. 15.—DRUM SCREEN AT TRIER. FRONT VIEW.

5.25 ft. (1.60 m.) wide. As at Gleiwitz, the free opening is 0.12 in. (3 mm.). Only one screen is operated at a time.

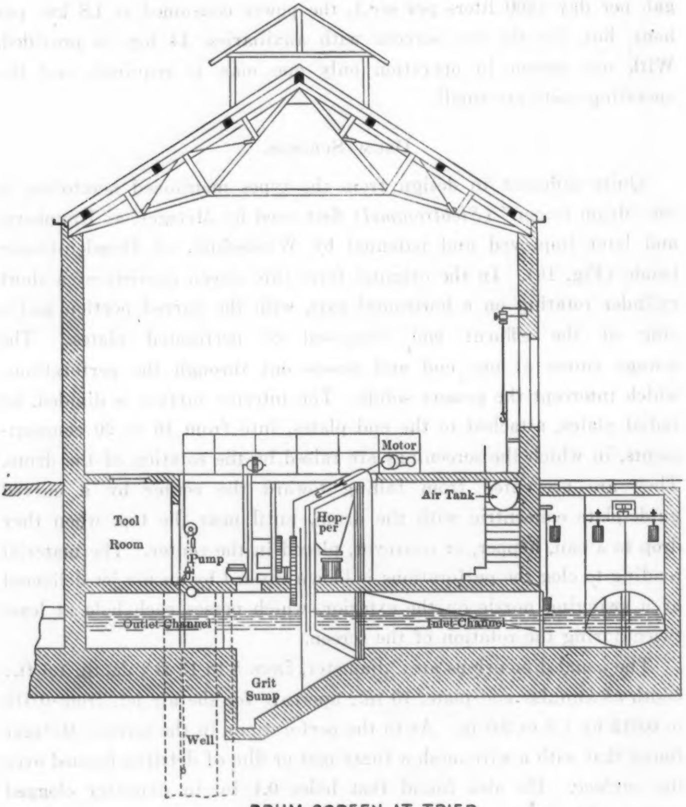
From 4 to 8 cu. yd. (3 to 6 cu. m.) of screenings, containing from 60 to 70% of moisture, are removed daily. With a flow of 4 600 000 gal. per day (200 liters per sec.), the power consumed is 1.8 kw. per hour, but, for the two screens with auxiliaries, 14 h.p. is provided. With one screen in operation only one man is required, and the operating costs are small.

Drum Screens.

Quite different in design from the types mentioned heretofore is the "drum screen" (*Siebtrommel*) first used by Metzger, at Bromberg, and later improved and patented by Windschild, of Dresden-Cossebaude (Fig. 16). In the original form this screen consists of a short cylinder rotating on a horizontal axis, with the curved portion and a ring at the effluent end composed of perforated plates. The sewage enters at one end and passes out through the perforations, which intercept the grosser solids. The interior surface is divided, by radial plates, attached to the end plates, into from 16 to 20 compartments, in which the screenings are raised by the rotation of the drum. They are prevented from falling toward the center by a curved guide-plate concentric with the drum, until near the top, when they drop to a can, hopper, or conveyor, placed in the center. The material tending to clog the perforations is blown inward by an air jet delivered by a swinging nozzle on the exterior, which passes each hole at least twice during the rotation of the screen.

The usual dimensions are: diameter, from 8 to 13 ft.; depth, 1.6 ft.; width of annular end plate, 10 in.; aperture for the air jet, from 0.010 to 0.012 by 1.2 to 2.0 in. As to the perforations in the screen, Metzger found that with a wire mesh a fuzzy mat or film of detritus formed over the surface. He also found that holes 0.4 in. in diameter clogged more readily than smaller ones. Both water and steam were tried for cleaning, but were found objectionable by increasing the moisture in the screenings. Steam, too, resulted in an objectionable odor. Compressed air gave the best results, but, to prevent the dissemination of spray or mist throughout the room, the jet should be protected by a box-like covering.

Bromberg.—At Bromberg there are four screens. Their diameter is 8.2 ft., and that of the perforations, 0.08 in. (Fig. 17). The screen rotates once in 50 sec., giving a rim velocity of 6½ in. per sec. The



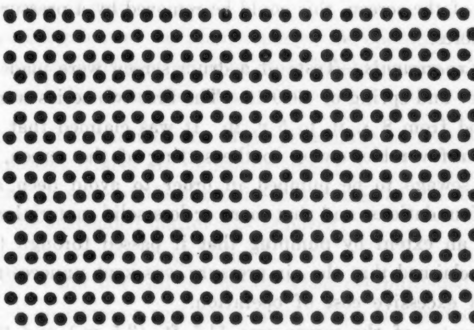
DRUM SCREEN AT TRIER

FIG. 16.

nozzle swings back and forth 85 times per min. With a slit 2 in. long, this requires 1 235 cu. ft. of free air, or from 70 to 77 cu. ft. of compressed air per hour. This has been considered too great, and the design is subject to improvement.

The screenings are removed by a band, 2 ft. wide. This machine screens 1 160 000 U. S. gal. per day, and $4\frac{3}{4}$ short tons (rising to a maximum of $10\frac{3}{4}$ tons) of material per million gallons are removed. This material contains from 40 to 60% of moisture. After drying on a warm plate to 34.3%, the weight is reduced to 7 short tons. Experiments lasting 8 days with two of these screens showed that the power required per screen was:

For turning the screen........	0.9 kw. per hour.
For compressing air..........	0.3 " " "
Total	1.2 kw. per hour.



2–mm. PERFORATIONS FOR
DRUM SCREEN AT BROMBERG

Fig. 17.

The two screens averaged 1 195 620 gal. per day, and, on the last day, one screen cared for 64 192 gal. in 1 hour. Only one attendant was required during the operation. The cost for wages and power amounted to \$2.45 per million gallons. If both screens were operated at a full capacity of 23 250 000 gal. per day, this cost would be reduced to \$1.09 per million gallons.

The screen-house, with deep foundations, cost \$15 600, and the mechanical plant, complete, \$7 450.

One of the improvements introduced by Windschild was the use of a truncated cone, instead of a cylinder, for the drum, this form being better adapted to fluctuations of flow. During periods of least flow,

only the conical surface comes in contact with the current, and in wet weather, the sewage passes through both the cone and the annular end plate. Windschild also perfected the form and application of the air jet. A fair velocity of the perimeter of the screen is 4 in. (100 mm.) per sec. The air pressure necessary for cleaning varies from 0.5 to 2.0 atmospheres, but, ordinarily, 0.6 atmosphere is sufficient, increased to 0.8 or 1.0 atmosphere when the load is great. One nozzle requires 700 cu. ft. (20 cu. m.) of air per hour. In operation this may be cut off for considerable periods during times when the sewage is small in volume or weak. After operating 4½ hours without air, the screen was well cleaned in a few rotations by raising the pressure to 1½ atmospheres, and, even if a continuous film was allowed to accumulate over the surface of the screen, this could be removed by a pressure of 1.5 to 2.0 atmospheres.

Mainz Experiments.—Tests of a drum screen were made at Mainz in the winter and spring of 1910-11. The first two series were confined to the period from 8 A. M. to 5 P. M. It was claimed that the conditions were unfavorable, owing to the setting of the screen, which required the sewage to be pumped in order to avoid deep and costly temporary foundations, and that more or less solid material was broken up to such an extent by pumping that it passed through the screen. It was also claimed that the air pressure was kept unnecessarily high, resulting in excessive cost of operation.

The diameter of the screen was 11.5 ft. (3.5 m.) and that of the perforations 0.12 in. (3 mm.); the width of the conical band was 2.73 ft. (0.83 m.), and that of the annular plate at the end, 1.32 ft. (0.52 m.). The total screen area was 103 sq. ft. (9.6 sq. m.), 55% of which was free opening. The normal capacity was 1.75 to 2.8 cu. ft. per sec. (50 to 80 liters per sec.) with a submerged surface of from 13.4 to 14.5 sq. ft. (1.24 to 1.34 sq. m.). The air pressure at the nozzles was 2 atmospheres. The total power provided was a 5-h.p. motor. As one revolution required 110 sec., the velocity of the periphery was about 4 in. per sec.

The power required to operate the drum alone was, in the first experiment, 1.5 to 1.9 kw. per hour, and in the second, 2.0 kw. per hour. For running the compressor, 2.4 kw. per hour were consumed in the first experiment, compressing to 1 atmosphere, and 3.2 kw. per hour in the second, compressing to 1.5 atmospheres.

In the third experiment one-quarter of the drum was covered with brass plates having 0.04 by 0.08-in. (1 by 2-mm.) slots and three-quarters with copper wire with 0.04, 0.06, and 0.08-in. (1, 1½, and 2-mm.) mesh. These results were more favorable as to power required, but indicated the desirability of some alterations of design.

The results in detail are given in Table 5*.

In general, it may be said that, notwithstanding the unfavorable conditions mentioned, and with a sewage containing barely 1 ton of suspended matter per million gallons (0.109 kg. per cu. m.), the screen removed from 46 to 61% of the solids, or from 0.46 to 0.52 cu. yd. per million gallons. The screenings contained about 75% of moisture. The cost ranged from 89 cents to $3.42 per million gallons, decreasing with an increase of flow.

The superintendent of the plant reported the operation of the screen as cleanly and without spattering or objectionable spray or mist.

It was concluded from these tests:

1.—That, on account of the large percentage of free opening, the exposed surfaces are unusually small;

2.—That, in spite of the limited exposed area of the screen, 53.5% of the suspended matter was removed;

3.—That the screen never clogged;

4.—That it did not fail to operate when overburdened;

5.—That no slime or rust spots appeared on the surfaces;

6.—That the cost of power was high, but that this was due in part to unfavorable conditions, and that this drawback would usually be offset by the small cost of installation for a given capacity.

Trier.—A drum screen was installed at Trier (Figs. 15 and 18) in December, 1912, to take the place of tanks. The latter were found costly to operate on account of difficulty in disposing of the sludge. The sanction of the Government to make the change was obtained, with the proviso that particles more than 0.06 in. (3 mm.) in diameter be removed.

The population of Trier is about 40 000. The water supply is 32 gal. (120 liters) per capita, and the sewage flow amounts to 1 270-000 gal. (4 800 cu. m.) daily, half of which runs off in 9 hours, or

* "Siebtrommel mit Druckluftreinigung," by Zivilingenieur G. Windschild.

TABLE 5.—(*Continued.*)

AVERAGES FOR FLOWS OF FROM 2.12 TO 7.94 CU. FT. PER SEC.

Date.	Hours in operation.	Volume of Sewage. Cubic feet per second.	Volume of Sewage. In gallons per day.	Screenings. Per Day. Pounds.	Screenings. Per Day. Cubic yards.	Screenings. Per million gallons. Pounds.	Screenings. Per million gallons. Cubic yards.	Deposit in Grit Chamber. Pounds.	Deposit in Grit Chamber. Cubic yards.	Loss of head, in feet.	Power consumed, in kilowatt-hours per day.	Cost. Power.	Cost. Wages.	Cost. Total.	Cost. Per million gallons.	Hours compressor out of service.	Remarks.
Nov. 22 to	4	2.12	228 000	232	0.140	1 058	0.594	......	....	0.111	15.8	$0.452	$0.333	$0.785	$3.42	....	
Dec. 3, 1910	45	2.82	685 500	2 960	1.565	876	0.445	......	....	0.154	176.7	5.042	3.750	8.792	2.52	....	
and																	
Jan. 16 to	61	4.94	1 208 000	5 300	3.990	650	0.445	......	....	0.098	217.1	6.202	5.080	11.282	1.35	....	
Feb. 1, 1911	15½	7.94	1 763 000	3 510	2.125	1 050	0.594	......	....	0.597	80.5	2.300	1.292	3.592	1.08	....	

TESTS FROM APRIL 20TH TO APRIL 25TH, 1911.

Date.	Hours in operation.	Cubic feet per second.	In gallons per day.	Per Day. Pounds.	Per Day. Cubic yards.	Per million gallons. Pounds.	Per million gallons. Cubic yards.	Grit Chamber. Pounds.	Grit Chamber. Cubic yards.	Loss of head, in feet.	Power consumed, in kilowatt-hours per day.	Power.	Wages.	Total.	Per million gallons.	Hours compressor out of service.	Remarks.
April 20, 1911	7	2.82	685 500	992	0.590	1 860	1.088			0.046	19.8	$0.568	$0.583	$1.151	$2.07	1½	*
" 21, "	10	4.94	1 208 000	2 355	1.312	1 768	0.990			0.174	36.2	1.033	0.833	1.866	1.35	⅔	
" 22, "	7	7.94	1 763 000	2 485	1.317	1 598	0.847			0.880	27.4	0.783	0.583	1.366	0.89	⅓	
" 24, "	10	2.82	685 500	1 644	0.881	2 160	1.138	4.06	0.202	0.043	36.3	1.037	0.833	1.870	2.43	½	
" 25, "	4	4.94	1 208 000	1 274	0.587	2 384	1.188			0.390	16.6	0.474	0.333	0.807	1.44	⅙	
" 25, "	6	2.82	685 500	1 405	0.789	3 069	1.682			0.239	21.8	0.624	0.499	1.123	2.43	½	

*The screen was covered: ¼ with brass plate, openings 2 by 1 mm.; ¼ with copper wire, 1-mm. mesh; ¼ with copper wire, 1½-mm. mesh; ¼ with copper wire, 2-mm. mesh.

AVERAGES FOR FLOWS OF FROM 2.82 TO 7.94 CU. FT. PER SEC.

Date.	Hours in operation.	Cubic feet per second.	In gallons per day.	Per Day. Pounds.	Per Day. Cubic yards.	Per million gallons. Pounds.	Per million gallons. Cubic yards.	Grit Chamber. Pounds.	Grit Chamber. Cubic yards.	Loss of head, in feet.	Power consumed, in kilowatt-hours per day.	Power.	Wages.	Total.	Per million gallons.	Hours compressor out of service.	Remarks.
April 20, 1911	23	2.82	685 500	4 040	2.260	2 300	1.286	......	....	0.108	77.9	$2.225	$1.915	$4.140	$2.34	....	
to	14	4.94	1 208 000	3 630	1.957	2 943	1.038	......	....	0.282	52.8	1.509	1.166	2.675	1.35	....	
April 25, 1911	7	7.94	1 763 000	2 485	1.317	1 592	0.847	......	....	0.880	27.4	0.783	0.583	1.366	0.89	....	

TABLE 5.—RESULTS OF TESTS OF WINDSCHILD'S DRUM SCREEN AT MAINZ,

FROM NOVEMBER 22D TO DECEMBER 3D, 1910.

Date.	Hours in operation.	Volume of Sewage. In cubic feet per second.	Volume of Sewage. In gallons per day.	Screenings. Per Day. Pounds.	Screenings. Per Day. Cubic yards.	Screenings. Per Million Gallons. Pounds.	Screenings. Per Million Gallons. Cubic yards.	Deposit in Grit Chamber. Pounds.	Deposit in Grit Chamber. Cubic yards.	Loss of head, in feet.	Power consumed, in kilowatt-hours per day.	Cost. Power.	Cost. Wages.	Cost. Total.	Cost. Per million gallons.*	Hours compressor was out of service	Remarks.
Nov. 22, 1910.	7	4.94	931 980	1 107.0	0.892	1 185	0.942			......	31.0	$0.885	$0.583	$1.468	$1.53	....	
" 23, "	8	4.94	1 065 120	1 281.0	0.936	1 202	0.842			......	31.0	0.885	0.666	1.551	1.44	....	
" 24, "	6	4.94	798 840	638.0	0.425	798	0.495	560	0.254	0.059	26.1	0.617	0.500	1.117	1.35	....	
" 25, "	3	4.94	399 420	488.0	0.313	1 168	0.743			0.131	15.3	0.438	0.250	0.688	1.71	3¼	
" 28, "	8	4.94	1 065 120	702.0	0.576	651	0.495			0.115	26.3	0.752	0.666	1.418	1.26	2½	
" 29, "	8	4.94	1 065 120	333.0	0.268	309	0.248			0.049	25.9	0.740	0.666	1.406	1.26	3½	
" 30, "	8	4.94	1 065 120	39.7	0.0	33	0.015	384	0.173	0.016	19.7	0.562	0.666	1.228	1.08	4½	
Dec. 1, "	7	4.94	931 980	38.6	0.016	33	0.020			0.016	20.0	0.571	0.583	1.154	1.17	2	
" 3, "	6	4.94	798 840	690.0	0.546	860	0.644			0.512	21.8	0.623	0.500	1.123	1.35	....	Much material due to cleaning sewers.

TESTS FROM JANUARY 16TH TO FEBRUARY 1ST, 1911.

Date.	Hours in operation.	In cubic feet per second.	In gallons per day.	Screenings per day, Pounds.	Screenings per day, Cubic yards.	Screenings per million gallons, Pounds.	Screenings per million gallons, Cubic yards.	Grit chamber, Pounds.	Grit chamber, Cubic yards.	Loss of head, in feet.	Power consumed, kilowatt-hours per day.	Power.	Wages.	Total.	Per million gallons.*	Hours compressor was out of service	Remarks.
Jan. 16, 1911.	5	2.82	380 500	186	0.089	484	0.198			0.023	16.0	$0.453	$0.416	$0.869	$2.25	1½	
" 17, "	8	2.82	553 000	415	0.173	742	0.297			0.298	33.1	0.945	0.667	1.612	2.88	1	
" 18, "	9	2.82	686 000	640	0.365	927	0.495			0.095	39.9	1.140	0.750	1.890	2.70	....	
" 19, "	8	4.94	1 208 000	......	......	......	......			......	......	......	......	......	......	....	
	1	2.82	114 200	772	0.441	675	0.346	850	0.356	0.082	38 6	1.102	0.750	1.852	1.62	¼	Detritus contained much hair, etc.
" 20, "	4	4.94	1 208 000	......	......	......	......			......	......	......	......	......	......	....	
	3	2.82	761 000	1 425	0.830	187	1.090			0.095	36.5	1.042	0.583	1.625	2.07	....	Operation suspended after 2.20.
" 21, "	4	2.82	304 500	704	0.427	2 300	1.385			0.089	23.9	0.686	0.333	1.019	3.33	....	Stopped for lack of load.
" 24, "	5	2.82	380 500	459	0.284	1 200	0.742			0.049	18.1	0.516	0.416	0.932	2.43	....	
" 25, "	9	2.82	685 500	684	0.363	998	0.495			0.292	34.7	0.990	0.750	1.740	2.52	2¼	Interrupted from 8.50-9.30 for repairs to motor.
" 26, "	9	2.82	685 500	581	0.293	843	0.396			0.177	34.9	0.995	0.750	1.745	2.88	2¼	
" 27, "	4	2.12	228 000	243	0.140	1 059	0.593	490	0.368	0.112	15.8	0.452	0.333	0.785	3.42	1¼	Stopped in P. M. on account of excessive friction.
" 28, "	9	4.94	1 208 000	1 104	0.678	918	0.544			0.272	40.4	1.154	0.750	1.904	1.53	½	Friction increases.
" 30, "	2	4.94	1 208 000	......	......	......	......			......	......	......	......	......	......	....	
	7	7.94	1 763 000	1 954	1.159	1 101	0.693			0.804	47.5	1.357	0.750	2.107	1.17	....	Detritus becomes slimy.
" 31, "	6½	7.94	1 892 000	1 422	0.886	1 018	0.593			0.604	35.2	1.004	0.543	1.547	1.08	....	
Feb. 1, "	9	7.94	1 927 000	2 170	1.238	1 076	0.643			0.590	45.8	1.295	0.750	2.045	0.99	....	

* Add for lubricants and polish 4½ cents per million gallons.

at a rate of 2.6 cu. ft. (75 liters) per sec. The maximum rate of flow is 28 cu. ft. (800 liters) per sec., as determined by gaugings.

The diameter of the screen is 14.5 ft. (4.40 m.) and its width is 3.94 ft. (1.2 m.). The perforations are 0.10 in. (2½ mm.) in diameter and spaced so that only 0.04 in. (1 mm.) of metal remains between the holes. The screen plate is 0.05 in. (1.2 mm.) thick. The screen rotates once in 3 min.

There are sixteen radial plates which raise the coarser screenings and drop them into a central hopper. The fine material which remains on the screen is blown off by three swinging nozzles, the motion of which covers all portions that have been immersed. From the hopper the screenings are delivered to large perforated buckets from which, after draining, they are emptied.

The screen is operated by a 1-h.p. electric motor. A 2½-h.p. motor operates the compressor, ½ h.p. being required to swing the nozzle arms and an equal amount to circulate the cooling water in the compressor. The air is compressed to 2 atmospheres and delivered to a receiver, from which it is led to the nozzles by a pipe and hose.

The screen removes from 0.39 to 0.42 cu. yd. (300 to 400 liters) of material per day; 2.03 tons (3 650 kg.), or about 5.2 cu. yd. (4 cu. m.), have been removed in 11 hours. As this contains only from 50 to 60% of water, it is not sludgy in character, and can be handled easily. It is sold to market gardeners for 14½ cents per cu. yd.

The operation, according to Engineer T. Schürmann, has been found faultless, and no trouble has been experienced from spray or odor.

The total cost of the plant, including buildings, machinery, connections with the sewer, etc., was about $8 600 (36 000 Marks). The cost of operation, exclusive of interest and sinking fund charges, is given in Table 6.

TABLE 6.—COST OF SCREENING PLANT AT TRIER.

Power, 4.32 kw-hr., 11 hr. for 365 days = 17 265 kw. @ 0.12⅛ Mark	= 2 158	Marks	=	$513.60
One laborer @4.20 Marks per day	= 1 533	"	=	364.85
One helper @2.00 " " "	= 803	"	=	191.10
Lubricating oil, waste, and grease @ 0.40 Mark per day	= 146	"	=	34.75
Miscellaneous	60	"	=	14.30
Total cost	4 700	Marks	=	$1 118.60
Deduct value of screenings sold	950	"	=	226.10
Net annual cost	3 750	Marks	=	$892.50

The price paid by neighboring farmers for the screenings is 0.80 Mark per cu. m. = 15 cents per cu. yd. At 1 270 000 gal. per day,

FIG. 18.—DRUM SCREEN AT TRIER. REAR VIEW.

the cost of operation, therefore, is $\frac{1\,118.60}{365 \times 1.27} = \$2.41$ per million gallons.

Drum screens have also been installed at Insterberg, Worms, Minden, Wartzen, and Osnabrück.

Osnabrück.—Osnabrück is a town of 83 000 inhabitants, 65 000 of whom are served by sewers. Only 17 000 are provided with water-closets. The normal volume of sewage is 1 600 000 gal. (6 000 cu. m.) per day, about 60% of which is collected by newly laid sewers, on the separate system, and pumped, the remainder being collected by combined sewers and delivered by gravity.

The screen is 16.4 ft. (5 m.), converging to 11.48 ft. (3.5 m.) in diameter, and 4.27 ft. (1.3 m.) deep. The perforations are 0.08 in. (2 mm.) in diameter. The maximum capacity of the screen is 33 cu. ft. (950 liters) per sec., with a temporary increase of water during storms, or the sewage from 90 000 persons.

The drum makes a revolution in from 2½ to 3 min. It is cleaned by two compressed air nozzles sweeping the conical portion and one sweeping the down-stream end, each swinging 75 times per min.

The power required for operation is about as follows:

For the screen	2.5 h.p.
For the compressor	5.0 "
For moving air jets	1.5 "
	9.0 h.p.

This is supplied by an alternating-current motor at 380 volts and 24 amperes.

Under ordinary conditions, the force consists of one man, but, during storms, an additional man is required.

The screenings amount to from 5¼ to 6½ cu. yd. (4 to 5 cu. m.) daily, or from 3¼ to 4 cu. yd. per million gallons of sewage, besides which, 0.65 cu. yd. (0.5 cu. m.) of grit is taken each day from a grit chamber placed behind the screen.

At the time the plant was inspected, some of the perforations were stopped by wool fiber, but these were readily removed with a jet of water. Otherwise, the cleaning device appeared to operate perfectly, and the authorities are so well pleased that a second screen is to be installed soon.

The cost of the one screen, with grit elevator and motors, was

$3 570 (15 000 Marks), but the entire works, including two sewage pumps, air compressor, motor, a Diesel motor for emergency use, heating plant, an independent water supply, building, shower baths, etc., cost $59 500 (250 000 Marks).

The Weand Screen.—A modification of the drum type of screen is found in that invented by O. M. Weand, and now installed at Reading, Pa.,* Atlanta, Ga., Baltimore, Md., and Brockton, Mass. This screen (Fig. 19) consists of a cylindrical frame having a horizontal axis and covered with a fine wire mesh, which is protected by an outer coarse mesh of No. 12 wire. The sewage enters by and overflows from a channel at one end, and passes through the wire mesh to an outlet channel below. The material retained is worked to the farther end by the rotation of the screen, being carried forward by an interior spiral flange. At the farther end, it is lifted by short radial plates nearly to the top, and then falls to a chute delivering to a belt or trough.

Reading, Pa.—The Reading screen (Fig. 19) is 12 ft. long, 6 ft. in diameter, and is covered with Monel metal mesh having about 36 meshes per in. This fine mesh is protected by a $\frac{3}{8}$-in. screen of No. 12 copper wire. It is cleaned by $\frac{1}{16}$-in. water jets, 8 in. apart, from a pipe which swings back and forth longitudinally near the outer surface. Screened sewage instead of water was found to answer very well for this purpose, but crude sewage caused frequent stoppages. The screen rotates about 8 times per min.

The sewage flow amounts to 5 000 000 or 6 000 000 gal. per day, but the capacity of the screen was stated by Mr. Weand to be not less than 8 000 000 gal. per day.

Mr. Kuichling estimated† the dried matter removed at 750 lb. per million gallons, or 90 parts per million out of 215 in the raw sewage, corresponding to an efficiency of 42 per cent.

The screenings amount to about 1 cu. yd. (20 to 35 cu. ft.) per million gallons, and have been analyzed as follows:‡

Moisture	89.5 %
Mineral matter	2.8 "
Volatile "	7.7 "
	100.0 %

* The Reading screen is not now in use.
† "Notes on Sewage Disposal," E. Kuichling.
‡ "Sewage Sludge," by Elsner, Spillner, and Allen, p. 208.

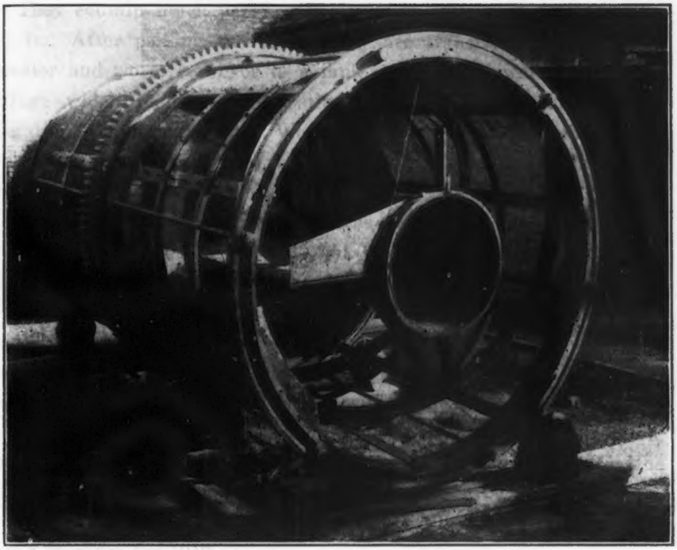


FIG. 19.—WEAND SCREEN PARTLY CONSTRUCTED. INLET END.

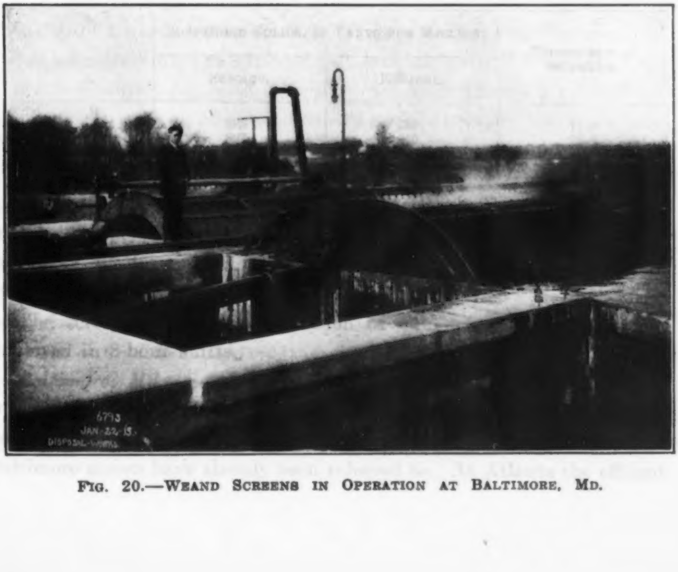


FIG. 20.—WEAND SCREENS IN OPERATION AT BALTIMORE, MD.

They contain much paper pulp, and weigh from 65 to 70 lb. per cu. ft. After passing the screen they are transferred by a bucket elevator and worm conveyor to a tank and then drawn off to a centrifuge which reduces the moisture to 73 per cent. The cost of operation was stated by Mr. Weand to be about $1 per million gallons. Within the past year or so, the use of this screen has been discontinued.

Brockton, Mass.—At Brockton, the sewage is from a separate system, and amounts to 1 750 000 gal. per day, or 40 gal. per capita. It is stronger, therefore, than the average sewage of American cities. The screen is similar to that at Reading, but several mechanical details have been improved. No trouble has been experienced from clogging, except by grease, and, to remove this, the screen is stopped for ½ hour every night and washed with caustic soda. Weak points in the design appear to be lack of proper support for the fine mesh, corrosion of the interior spiral plate, and wear in the holes for the bolts securing the frames to which the mesh is attached.

Table 7 indicates the efficiency of the Brockton screen, the figures being averages for 1912.

TABLE 7.—EFFICIENCY OF BROCKTON SCREEN.

	SUSPENDED SOLIDS, IN PARTS PER MILLION.		Percentage of reduction.
	Sewage.	Effluent.	
Total	983	282	71.3
Organic	897	245	72.7

During 1912, the average weight removed per day was 4 891 lb., or 1.4 tons per million gallons. In the first week of February of that year 4.5 tons, or 8 cu. yd. of screenings were removed from 2 000 000 gal. of sewage.

The screen required the attention of one man, three men being employed in 8-hour shifts.

Baltimore, Md.—Screens of the Weand type have been installed at Baltimore, Md. (Fig. 20), and Atlanta, Ga., primarily to prevent the clogging of the filter nozzles. The good results obtained by the Baltimore screen have already been referred to. At Atlanta the effluent

from the Emscher tanks has been so satisfactory that, as already stated, further cleaning by screening before application to the filters is not required.

Drum screens can receive the grit as well as true screenings, and, therefore, do away with the cost of grit chambers. They operate under varying loads at speeds that may be adjusted accordingly. They are compact, operate continuously, without hand labor, and the screenings, when cleaned by air, are delivered with a relatively low moisture content.

It is claimed for the Windschild screen, that there is practically no leakage past the outside of the drum, that losses from friction and deterioration are small, and that there is little backing up of the sewage by their use.

In the Weand screen, as installed at Reading and Brockton, there is a loss of several feet of head in the passage through the screen. Friction of the bearings originally gave much trouble at Baltimore, but this has been overcome by recent alterations. At Brockton, the interior flanges and the screen frames have shown considerable deterioration.

The Riensch-Wurl Screen.

Following the permission granted to the City of Dresden in 1903, to discharge sewage into the Elbe, if material greater than 3 mm. in diameter was removed, a novel type of screen, devised by Riensch, was installed. Since that date the inventor has died and the patent rights have been transferred to Wilhelm Wurl, of Berlin-Weissensee, who has made important improvements on the original design. Because of the very interesting and well-operated plant at Dresden, this screen has attracted much attention, and has already been adopted by a number of cities, including Bremen, Christiania, Stettin, Karlsruhe, Mainz, Strassburg, St. Petersburg, Toulon, and Astrachan.

In its present form the Riensch-Wurl screen (Fig. 21) consists of an annular disk surmounted by a truncated cone, both of which are perforated with slots from 0.04 to 0.20 in. (1 to 5 mm.) in width.* In general, the shape may be compared to that of a hat with a flat brim. This is mounted and rotates on a shaft inclined from 10 to 30° from the vertical, and is partly submerged. As a result, the floating solids are raised gently above the surface by the

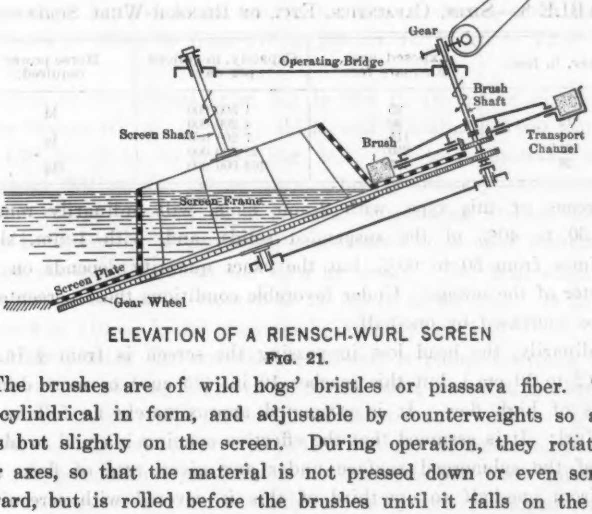
* It is claimed that ½-mm. slots are practicable.

rotation of the screen, and while in this position are brushed off to a container or a conveyor by brushes revolving on arms.

As the channel conforms to the rim of the disk, it is curved in section, and, therefore, there is a smaller variation in level for a given range of discharge than with a rectangular section.

Under ordinary conditions, the cone remains above the surface and so does not require the operation of the special brush provided to clean it, but, as the flow increases, this, too, is brought into service. The speed of rotation can be varied with the flow.

The screen plates are of brass or bronze. The perforations are beveled so that material which has once entered the slot will pass through without clogging.



ELEVATION OF A RIENSCH-WURL SCREEN
FIG. 21.

The brushes are of wild hogs' bristles or piassava fiber. They are cylindrical in form, and adjustable by counterweights so as to press but slightly on the screen. During operation, they rotate on their axes, so that the material is not pressed down or even scraped forward, but is rolled before the brushes until it falls on the conveyor or into the receptacle provided for it. By this means there appears to be no clogging of the holes except by a gradual deposit of grease, which is removed every few months by a jet of dry steam. The brushes themselves are cleaned by passing over a comb at each revolution. The speed and arrangement of brushes is such that each part of the screen is cleaned four or five times before being again submerged.

A "service bridge" is constructed across and above the screen, to which are attached the ball-bearings on which the screen frame is hung. There is only one bearing that is submerged, and this serves as a guide for the lower end of the shaft. It is protected by a stuffing-box lubricated by oil under pressure, and no trouble has been experienced as yet from the introduction of grit or other foreign material. The ball-bearings run in oil, but slow-moving parts are lubricated by grease.

One feature peculiar to the Riensch-Wurl screen is the large percentage of surface available for actual use.* The approximate area exposed to the sewage, the capacity and power required for screens of different sizes but with slots 2 mm. wide, are as given in Table 8.

TABLE 8.—SIZES, CAPACITIES, ETC., OF RIENSCH-WURL SCREENS.

Diameter, in feet.	Exposed area, in square feet.	Capacity, in gallons per day.	Horse-power required.
6	25	1 800 000	1/4
12	90	4 200 000	
14	115	7 200 000	1/2
20	250	28 800 000	
26	405	103 000 000	2 1/2

Screens of this type, with 2-mm. slots, will ordinarily remove from 30 to 40% of the suspended solids, and, with 1-mm. slots, sometimes from 50 to 60%, but the exact quantity depends on the character of the sewage. Under favorable conditions these percentages may be increased by one-half.

Ordinarily, the head lost in passing the screen is from $\frac{3}{4}$ in. to 4 in. (2 to 10 cm.), but this reaches 10 in. (25 cm.) or more during periods of high flow. It is estimated approximately as follows by Mr. Wurl: It is assumed that the effective opening is equal to about 25% of the submerged surface under any given rate of flow, and that from one-half to one-third of this is covered with screenings and, therefore, useless.

Let S = the submerged surface, in square feet;

A = the true effective area = say, $0.25 \times 0.4\ S$, surface $= 0.1\ S$;

Q = the quantity of sewage, in cubic feet per second;

$G = 32.2$;

C = a coefficient provisionally assumed as 0.4.

* Claimed by the manufacturer to be 80%, as compared with 33⅓% for band screens and 20% for wing screens.

Then, the head lost, in feet, is:

$$H = \left(\frac{Q}{A\ C\ \sqrt{2\ G}}\right)^2 = \frac{Q^2}{10.3\ A^2} = 0.097\ \frac{Q^2}{A^2} = 9.7\ \frac{Q^2}{S^2}.$$

Dresden.—The Dresden plant (Figs. 22 and 23) consists of a grit chamber, a 2.65-in. (66-mm.) fixed-bar screen, and four Riensch-Wurl screens, the first of which was installed in 1910. The population served is about 530 000, but, of these, only 280 000 had water-closet connections in 1913. The average sewage flow is 26 500 000 gal., including industrial wastes, or 50 gal. per capita daily, but this varies from 18 to 21 cu. ft. (500 to 600 liters) per sec. between 3 and 6 A. M. to about 70 cu. ft. (2 000 liters) per sec. between 11 A. M. and 5 P. M. This flow is easily taken care of by one screen, the other three being held in reserve for use in storms, when the flow increases at times to from 175 to 260 cu. ft. (5 000 to 7 500 liters) per sec.

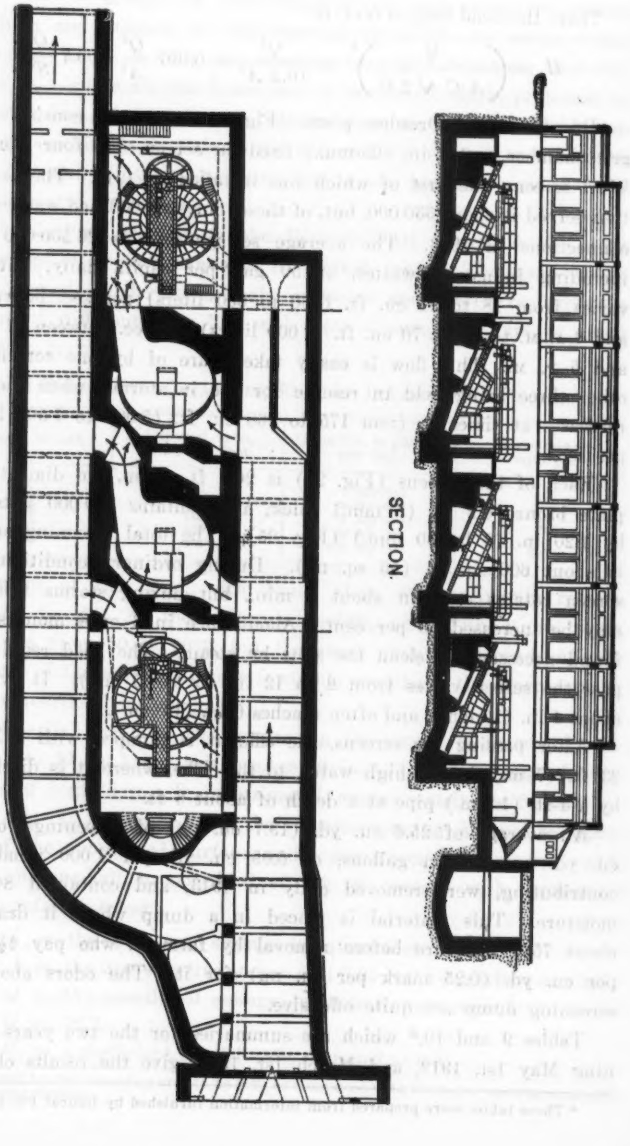
Each of the screens (Fig. 24) is 26.2 ft. (8 m.) in diameter, of plate bronze 0.2 in. (5 mm.) thick, and contains 230 000 slots 0.08 by 1.20 in. (2 by 30 mm.) (Fig. 25). The total screening surface is about 600 sq. ft. (56 sq. m.). During ordinary conditions, the screen rotates once in about 3 min., but during storms this rate may be increased 50 per cent. About once in 3 or 4 months it is found necessary to clean the slots by steam. The head required to pass the screen varies from 2 to 12 in. (5 to 30 cm.). It averages about 4 in. (10 cm.), and often reaches 6 in. (15 cm.).

After passing the screens, the effluent is pumped with a lift of 23 ft. (7 m.) during high water, to the Elbe, where it is discharged by a 4-ft. (1.2-m.) pipe at a depth of about 6 ft.

An average of 25.6 cu. yd. (19.7 cu. m.) of screenings, or 0.97 cu. yd. per million gallons, or 0.09 cu. yd. per 1 000 inhabitants contributing, were removed daily in 1913, and contained 84% of moisture. This material is placed in a dump where it drains to about 75% moisture before removal by farmers, who pay 4½ cents per cu. yd. (0.25 mark per cu. m.) for it. The odors about the screening dump are quite offensive.

Tables 9 and 10,* which are summaries for the two years beginning May 1st, 1912, and March 1st, 1913, give the results obtained

* These tables were prepared from information furnished by Baurat Fleck.



DRESDEN SCREENING PLANT

Fig. 22.

FIG. 23.—RIENSCH-WURL SCREEN CHAMBER AT DRESDEN.

FIG. 24.—RIENSCH-WURL SCREEN AT DRESDEN.

in regular operation. The efficiency is seen to vary between about 60 and 70% in the first series, and between 30 and 45% in the second series. This marked difference is accounted for by the mode of sampling, which is described. The power required varies from 2.5 to 3.0 kw.

The cost of the Dresden plant was about \$2 150 000; the screens themselves cost \$12 000 (50 000 Marks) each.* The screen-house is 197 by 37 ft. The screenings find a market among the neighboring farmers at about 9½ cents per cu. yd.

As a result of experiments in 1911, the City of Mainz decided to install a Riensch-Wurl screen similar to those at Dresden.†

The diameter of the screen is 14.8 ft. (4.5 m.) and the height of the cone 16 in. (40 cm.). The brass plates are perforated with holes 0.08 by 1.2 in. (2 by 30 mm.), as at Dresden. For flows of 2.1, 4.9, and 11.2 cu. ft. per sec. (60, 140, and 320 liters per sec.) the corresponding free openings in the part submerged are 7.2, 12.4, and 21.2 sq. ft. (2.52, 4.29, and 7.31 sq. m.). With rates less than 4.9 cu. ft. per sec. (140 liters per sec.), the cone with its cleaning brush was out of use.

The screen intercepted from 36 to 42% of the suspended matter, or an average of 0.395 ton per million gallons (0.095 kg. per cu. m.). There was little clogging of the slots, and the screen was kept quite clean; some rust spots appeared, however, on parts of the frame which were submerged. The costs of operation with different rates of flow are given in Table 11.

Comparison of Screens.

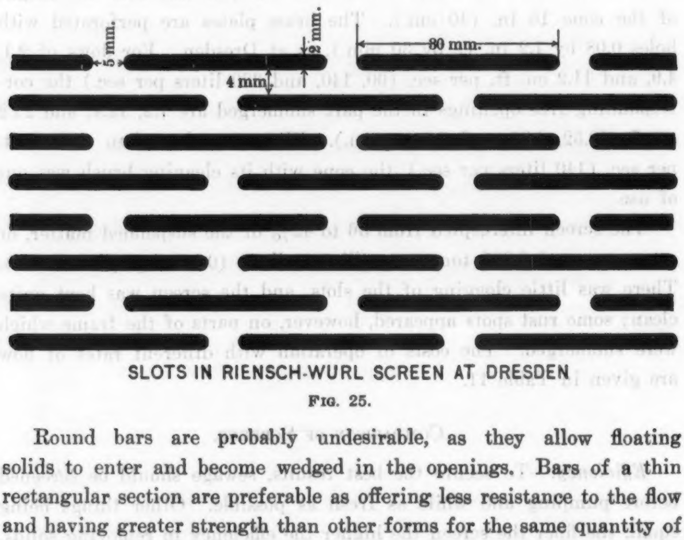
Efficiency.—To secure the best results, sewage should be screened before pumping and while as fresh as possible. Other things being equal, the finer the screen the higher the efficiency in removing solids, but other factors enter which, as will be seen later, often obscure this effect. For instance, unless the screen extends over the entire cross-section of the channel occupied by the stream of sewage, a certain quantity of this will be by-passed with its suspended solids. It is for this reason that at least five vanes are required with the wing or

* *Municipal Journal*, December 28th, 1911.

† *Engineering Record*, Vol. 67, p. 471.

shovel-vane type of screen, and an invert depressed so that every portion of the stream will have to pass through one vane. To insure against leakage past the Riensch-Wurl screen, adjustable plates are set in the channel close to the periphery of the screen plate.

The efficiency also depends on whether the screening surface consists of bars, perforated plates, or a wire mesh, whether the solid particles are broken up by abrasion due to too high velocities or are macerated, so as to be carried through the screen by the current, or whether they are pressed through in the process of cleaning. Velocities through the screen should be moderate, and for this reason it is better that the motion of the screen should not be against the current, and that it should be adjustable.



SLOTS IN RIENSCH-WURL SCREEN AT DRESDEN

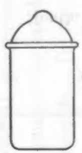
FIG. 25.

Round bars are probably undesirable, as they allow floating solids to enter and become wedged in the openings. Bars of a thin rectangular section are preferable as offering less resistance to the flow and having greater strength than other forms for the same quantity of metal. At Hamburg, the links are 0.20 in. (5 mm.) thick and are 0.6 in. (15 mm.) apart; at Frankfort and Elberfeld the bars are 0.12 in. (3 mm.) thick and are 0.4 in. (10 mm.) apart. Experiments made on wooden strips, 3 in. by ½ in. in section, at Cornell University* indicate a co-efficient of discharge of 0.811 if the sides are left parallel, increasing to 0.832 if sharpened on the down-stream edge, and becom-

* *Engineering Record*, Vol. 62, p. 762.

TABLE 9.—RESULTS OBTAINED BY OPERATION OF RIENSCH-WURL SCREENS AT DRESDEN, FROM MAY 1ST, 1912, TO APRIL 30TH, 1913.

Month.	Suspended Matter in Sample Before Screening. 4 A. M.	8 A. M.	12 M.	4 P. M.	8 P. M.	12 Mn.	Daily average, in parts per million.	Percentage of moisture in sample.	Time of operation. Hrs. Min.	Screenings, cubic yards.	Suspended Matter in Sample After Screening. 4 A. M.	8 A. M.	12 M.	4 P. M.	8 P. M.	12 Mn.	Daily average, in parts per million.	Percentage of moisture in sample.	Percentage of efficiency of screen.
May......	9 800	9 080	11 020	12 090	13 090	14 790	11 640	According to other extended experiments, assumed to average 97.8%.	23 29	26.40	2 660	2 620	3 760	3 770	3 470	5 080	3 560	According to other extended experiments, assumed to average 95.7%.	69.4
June.....	8 520	9 600	11 320	10 890	12 170	12 980	10 910		26 28	26.04	1 830	2 980	3 550	3 580	3 620	3 280	3 140		71.2
July.....	4 580	10 090	12 040	12 040	7 140	5 510	8 260		23 41	23.81	1 630	3 360	4 080	4 090	3 810	2 910	3 310		59.9
Aug......	9 860	8 950	12 760	13 500	13 880	13 210	12 020		25 14	24.35	2 680	3 060	3 460	4 120	4 140	4 600	3 670		69.4
Sept.....	8 740	7 830	12 270	12 120	13 480	12 040	11 080		23 47	26.49	2 770	3 240	4 380	4 870	4 320	4 520	4 010		63.8
Oct......	9 030	9 210	12 270	14 410	13 990	12 940	11 970		22 36	28.88	2 310	3 930	5 310	5 110	4 450	3 420	4 090		65.8
Nov......	11 140	12 400	17 010	15 150	14 620	13 270	13 930		23 17	25.81	3 700	3 270	4 600	4 780	5 000	4 810	4 360		68.7
Dec......	11 990	11 290	18 440	21 060	18 000	13 910	15 780		24 09	26.26	3 610	3 220	5 810	6 300	4 870	4 490	4 710		70.1
Jan......	9 770	7 280	11 160	13 140	13 600	13 700	11 440		22 45	23.15	2 520	2 100	3 880	4 470	4 350	3 550	3 470		69.6
Feb......	4 220	5 290	9 350	10 620	8 130	6 900	7 420		24 39	24.71	1 480	1 900	4 190	4 220	3 590	3 380	3 130		57.8
Mar......	7 890	4 840	9 890	9 260	8 690	8 280	8 060		23 19	23.15	2 200	1 430	4 280	4 260	4 040	3 110	3 220		60.0
Apr......	4 260	4 990	9 300	10 180	8 970	8 200	7 650		23 12	25.32	1 170	1 690	4 650	3 680	3 740	3 390	3 050		60.1
	8 270	8 400	12 230	12 720	12 140	11 310	10 840 a	97.8	23 53	25.35	2 880	2 730	4 330	4 440	4 110	3 880	3 640 b	95.7	$66.4 = \frac{a-b}{a}$

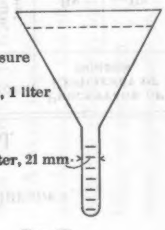


Sampler

Capacity, 1 liter

FIG. 26.

Six samples were examined daily. They were taken, from 4 A. M. to midnight, from a moderate depth below the surface, above and below the movable screen, by buckets containing 1 liter (Fig. 26). These samples were decanted into a glass funnel with a graduated stem, 21 mm. in diameter (Fig. 27). The settled sludge was removed every 8 hours. As the samples were imperfect, because of the small diameter of funnel stem and uncleaned vessels, this method was abandoned after a year's trial.



Measure

Capacity, 1 liter

Diameter, 21 mm.

FIG. 27.

TABLE 10.—SUMMARY OF RESULTS OF OPERATION OF RIENSCH-WURL SCREENS AT DRESDEN,
From March 1st, 1913, to February 28th, 1914.
Note: Fecal Matter from an Average of but Half the Properties Connected.

Month.	Suspended Matter in 10-Liter Samples of Sewage before Screening.						Time of operation of screen.	Volume of screenings removed, in cubic meters.	Suspended Matter in 10-Liter Samples of Sewage after Screening.						Percentage of Efficiency of Screen.	
	Volume wet, in cubic centimeters.	Weight of dry material, in grammes.	Loss by incineration, in grammes.	Residue after incineration, in grammes.	Sum of daily specific gravities.	Sum of daily moisture, per cent.	Hrs. Min.		Volume wet, in cubic centimeters.	Weight of dry material, in grammes.	Loss by incineration, in grammes.	Residue after incineration, in grammes.	Sum of daily specific gravities.	Sum of daily moisture, per cent.	By volume.	By weight.
Mar., Apr., May	1 153.0	50.9621	31.2458	19.7163	23.57	1 535.9	544 10	442.9	782.0	41.4030	24.4710	16.9320	23.67	1 526.6	32.17	18.75
June, July, Aug.	968.0	61.4275	33.6803	27.7472	33.08	1 915.1	550 15	415.3	663.5	50.7896	26.3830	24.4066	33.51	2 000.1	31.45	21.25
Sept., Oct., Nov.	1 257.0	81.7181	47.2796	34.4385	37.41	2 204.0	566 05	484.5	902.0	59.1381	33.6270	25.5111	37.16	2 206.5	28.24	27.68
Dec., Jan., Feb.	1 193.5	99.4181	55.6602	43.7579	30.84	1 809.8	548 30	435.3	686.0	51.3313	26.9992	24.3321	30.27	1 717.6	42.52	45.50
Total......	4 571.5	293.5258	167.8659	125.6599	124.90	7 464.8	2 209 00	1 778.0	3 033.5	202.6620	111.4802	91.1818	124.61	7 450.8		
Equivalent days of examination	85	79	79	79	77	78	90	90	85	79	79	79	77	78	$\frac{a-b}{a}$	$\frac{c-d}{c}$
Average.........	53.78	3.7155	2.1248	1.5906	1.62	95.7	24 33	19.7	35.68	2.5653	1.4111	1.1542	1.62	95.5	= 33.64	= 30.95
	a	*c*							*b*	*d*	As to organic material as shown by loss on incineration.........					33.59

Six samples were taken every fourth dry day at 4-hour intervals, from 2:30 A. M. until 10:30 P. M., at designated points above and below the screen by a "Hofer" apparatus (Fig. 28) containing 20 liters. The samples below the screen were taken on the average about 20 sec. later than those above, depending on the velocity. Each series of 6 samples collected at the two points was placed in a vessel holding 120 liters. From these vessels 10 samples of 1 liter each were drawn while being constantly stirred. The latter average daily samples from above and below the screen were poured into a tin funnel (Fig. 29) connected with a graduated glass cylinder 35 mm. in diameter and having a capacity of 150 cu. cm. All the sludge deposited within 4 hours was collected in the cylinder by jarring the funnel. It was then removed and the contents of the cylinder sent to the laboratory (after being rendered bacteriologically stable with chloroform) for further examination.

ing 1.032 if sharpened on both edges. The great advantage in the latter cannot be availed of, for the same reason that round bars are objectionable. In the shovel-vane screen at Gleiwitz, the curved bars taper on the down-stream side or toward the center of the screen. Aside from the small hydraulic advantage, this permits material once passing the up-stream surface of the vane to pass through freely.

Perforated plates, as in the Riensch-Wurl and Windschild screens, have an advantage in intercepting straws, matches, hair, and fiber which might pass between bars or even slots with equal spacing.

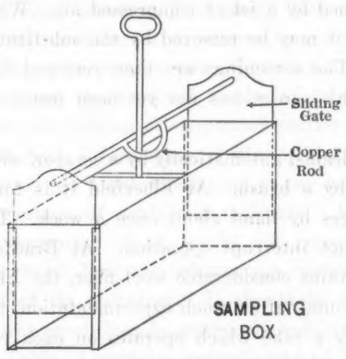


FIG. 28.

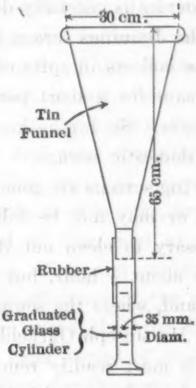


FIG. 29.

The American wire-mesh screens of Weand and Jennings are the finest of all, each mesh, including the wire, being only about 0.03 in. (0.75 mm.) square. Objection has been found with mesh screens in that they promote the disintegration of the solids, so that an appreciable quantity passes through with the liquids. This was observed by Watson in connection with a Carshalton screen at Birmingham, and in connection with the Riensch-Wurl screen, and has also been mentioned with reference to tests of the shovel-vane screen at Gleiwitz.

A very important feature in any screening device is the method of cleaning, as the thoroughness of this operation directly affects, not only the efficiency, but also the cleanliness of the plant and the reliability of operation. There are many ways of cleaning bar screens. In some, particularly in England, the fixed bars are cleaned by movable rakes, but as it is preferable that all cleaning be done above water, in

order to prevent the escape of broken-up particles with the effluent, it follows that the screen itself should raise the detritus to the proper elevation for removal. This is a feature of all the fine screens which have been described in detail herein.

The Hamburg screen is cleaned automatically by a hard-rubber rake or comb which engages the links after they have passed over the top, and is then withdrawn past a scraper which causes the screenings to fall on a belt conveyor. The motion of both rake and scraper is transmitted from a cam on the driving shaft by a system of levers. The device is carefully designed and appears to work satisfactorily.

The Jennings screen is cleaned by a jet of compressed air. Where grease collects in spite of this, it may be removed by the substitution of steam for a short period. The screenings are then removed by a conveyor. So far as known, this screen has not yet been tested out with domestic sewage.

Wing screens are generally cleaned automatically by a scraper, which may, or may not, be followed by a brush. At Elberfeld it is found necessary to clean out the spaces by hand about once a week. This takes about ½ hour, but does not interrupt operation. At Bradford, England, where the sewage contains considerable wool fiber, the Engineer, Mr. Joseph Garfield, has found, after much experimentation, that this is most readily removed by a rake which operates on each vane when brought to and held in a horizontal position.

The shovel-vane and drum screens have an advantage in the removal of the screenings chiefly by gravity, the residue only being removed by a brush or jet.

With the Riensch-Wurl screen, the material is brushed off the smooth plates by automatically operated brushes. As already mentioned, it has been claimed that this operation forces some of the material through the slots, but to the writer the removal appeared to be very complete. Once in 3 or 4 months, however, sufficient grease is deposited in the slots to necessitate its removal by a jet of superheated steam.

Metzger experimented with various methods of cleaning perforated drum screens, and concluded that compressed air was most efficient and inoffensive. It has the advantage of blowing the detritus into the drum and delivering it with less moisture than, for instance, occurs in

screenings from the Weand screen, where the mesh is cleaned by moving jets of water. Screenings from the Weand screen contain nearly 90% of moisture, but in those from the Windschild screen the quantity is only two-thirds as great. The reason for so much moisture in the Jennings screenings is not apparent. A further objection to cleaning by water jets is the disintegration of the solid material, allowing it to pass the screen.

The thoroughness of screening is best indicated by the parts per million of dried suspended matter removed from identical sewages. Some valuable data of this kind were contained in a paper by Emil Kuichling, M. Am. Soc. C. E., read before the New Jersey Sanitary Association in December, 1908, in which the material removed by grit chambers and screens is given in pounds of dry matter per million gallons of sewage, but such information is not often obtainable from the reported results of screening. If one cannot find the parts per million of suspended matter before and after screening, the next best information is the cubic yards (or tons) of screenings secured per 1 000 population, or per million gallons of sewage, and the percentage of contained moisture. A record of this, kept at every fine-screening plant, would not require great effort and would furnish most desirable information regarding the efficiency of the screen and of its operation. It should be borne in mind that the weight of wet screenings is materially reduced by a brief period of draining.

Table 11 contains general data of this nature for the several types of screens under consideration, which the writer has collected.

The results stated in Table 11 do not indicate the increased efficiency due to fineness of screen that one would naturally look for. More good experiments with different types of screens, and under identical conditions, are needed, such as those attempted in the comparative tests of the Windschild and Riensch-Wurl screens at Mainz. In general, the data collected show that from ½ to 1½ cu. yd. or more of screenings may be removed from 1 000 000 gal. of ordinary domestic sewage at a cost of from $1 to $2, depending on the conditions. Further experience will probably develop the fact that care in operation has much to do with the station efficiency of any fine screen. A case has been brought to the writer's attention where a change of administration was the apparent cause of a decrease of 16% in the annual efficiency. The importance of using the most approved method of

TABLE 11.—DATA RELATING TO THE SEVERAL

Type of screen.	Name of inventor or manufacturer.	Location.	Clear opening, in inches.	Screenings. Per million gallons.	Per 1 000 population daily.
Band......	Brunotte..........	Hamburg....	0.6	0.34 cu. yd.	0.018 cu. yd.
	Herzberg.........	Göttingen....	0.4	0.35 "	0.026 "
	John Smith & Co..	Sutton.......	0.375 meshes per inch.	0.6 ton	....
	Jennings	Chicago Stock Yards....	....	2.4-3.1 tons	....
Wing......	J. S. Fries Sohn....	Frankfort....	0.40	0.7 cu. yd.	0.040 cu. yd.
	" "	Elberfeld....	0.40	1.15 "	0.053 "
		Stralsund....	0.20	....	0.079 "
		Wiesbaden..	0.60	1.1 "	0.083 "
Shovel-vane...	Geiger Mach. Wks.	Strassburg..	0.10	1.6 "	0.043 "
	" "	Gleiwitz.....	0.12	....	0.192 "
	" "	Temesvar....	0.12	0.9-1.7 "	0.067-0.133 "
Drum......	Windschild........	Bromberg...	0.08	4¾ tons	....
	"	Mainz........	0.12	0.52 cu. yd.	....
	"	Trier........	0.04-0.08 0.10	0.39-0.42 "	0.13 cu. yd.
	"	Osnabrück...	0.08	3.2 -4.0 "	0.08-0.10 "
	Weand	Reading, Pa.....	(36 meshes per inch.)	1.0 "	....
	"	Brockton, Mass....	(36 meshes per inch.)	1.4 tons	....
Riensch-Wurl..	Riensch-Wurl.....	Dresden......	0.08	0.97 "	0.09 "

* The figures in this column should be used with caution. More complete data should

screens.

sampling has already been indicated with regard to the results obtained at Dresden.

It has been clearly demonstrated through experiments at Ithaca by Messrs. T. C. Schaetzle and E. R. Davis* with a ½-in. wire mesh screen, that the efficiency varies greatly with the strength of the sewage. At 4 A. M., 2.6 lb. of dry material were removed per million gallons, and at 11 A. M., 104 lb., with an average for the 24 hours of 52.7 lb.

To obtain true average efficiencies, the hourly efficiencies should be weighted according to the rate of flow, in a way similar to that by which composite samples of average water or sewage are prepared for analysis. This is a point, by the way, which appears to be generally neglected.

* *The Cornell Civil Engineer*, November, 1913.

SCREENS DESCRIBED IN THIS PAPER.

Percentage of moisture.	Percentage of efficiency.*	Horse-power per screen.	Cost of Operation. Per million gallons.	Cost of Operation. Per cubic yard of screenings.	Authorities.	Remarks.
87	......	2.5	....	......	Baudirektor Sperber.	After removal of half this volume of grit.
......	90 (?)	2.0	....	......	A. Frühling.	
......	......	....	....	......	E. Kuichling.	
79	63	....	....	......	C. A. Jennings.	
......	10	5.0	....	$0.18	Stadtbaumeister Schaefer.	After removal of 16% by grit chamber.
75	......	....	....	......	Dipl.-Ing. Sturmfels.	Including 0.6 cu. yd. grit per million gallons.
......	......	4.5	....	......	August Frühling.	
......	......	Hand power	$1.64†	......	Engineer Vogel.	After passing 1.6-in. bar screen.
89.3	10-12	3.35	....	0.054	Stadtbaurat Strobl.	After removal of 0.132 cu. yd. grit and coarse screenings per 1 000 population.
......	63	....	0.90	0.125	Magistrat ——	
60-70	......	....	Small		Stadt-Ing. Vidrighin.	
40-60	......	....	2.45	......	Stadtbauinspektor M. Knauff.	Experimental.
75	53.5	5.2-6.8	0.89-3.42	......		
50-60	......	....	2.41	......	Abt. Baum. Schurmann, G. Windschild.	" "
......	......	9.00	....	......	Dipl.-Ing. Schlüssel G. Windschild.	Before removal of 0.4 cu. yd. grit per million gallons.
89.5	42	2.0	1.00 ±	......	E. Kuichling. O. M. Weand, E. B. Ulrich.	
......	71.3				C. R. Felton, J. Hayes, Jr.	
84	33.6	2.5	0.325-1.76	......	Stadtbaurat Fleck, W. Wurl.	

be secured, in order to furnish a reliable comparison between the efficiencies of different

† Including coarse screening, settling, and subsequent screening.

The influence of pumping and of length of travel, in comminuting the solids and increasing the colloidal matter, should be kept in mind as factors which may cause a marked lowering of efficiency.

The effect of preliminary cleansing by coarse screens or grit chambers should properly be considered in passing judgment on the relative merits of different methods of fine screening.

Adequacy and Reliability.—All the fine screens which have been described depend on continuous motive power for operation, and, having this, they appear to be quite dependable within the ranges of sewage level for which they are designed. In the case of the band screen, the surface may rise nearly to the belt conveyor, which is usually just below the top; with the wing, shovel-vane, and drum

screen, it may rise to a point just below the axle, and, with the Riensch-Wurl screen, to the highest point reached by the base of the cone during a revolution. Larger proportions of band and Riensch-Wurl screens therefore are submerged during periods of maximum flow and less head room is required. As the range of level with a Riensch-Wurl screen is limited by the diameter and inclination, this type is not well suited to small volumes of flow combined with wide variations in level. Where subject to these conditions, the band screen offers advantages, as it may be of any length without appreciably affecting other dimensions or costs.

Operation should be automatic, as far as possible. It is nearly so in all these screens, except when the speed is to be varied or the screen is to receive a special cleaning. With speeds that are too low, clogging and backing up of the sewage may result. The fine deposit of grease that usually occurs on any screen will require special removal at times, and, with fine perforations or meshes, a mat of fine fiber or hair may gradually form over the surface; otherwise, the cleaning devices appear to be effective.

Reliability of operation depends, also, on the design and workmanship. The simpler the design the better, especially as this facilitates renewals and repairs. The wing screen and some forms of band screen are simpler perhaps than the others, but serious objections under this heading have been pretty well met in the design of the other types. In reference to this it may be stated that the Carshalton screen is usually operated by the hydraulic power of the sewage itself, and, therefore is not dependent on outside sources of motive power. The possibility of operation by hand in case of accident to machinery is a claim made for the wing screen, but might apply as well to the others, if of small size.

Nuisances.—If properly operated and kept clean, there need be no objectionable odors or other offensive conditions in the neighborhood of works using these screens. Inside the station there is usually the smell of fresh sewage, which is not strong nor likely to be carried far. With septic sewage, of course, there is more danger of foul odors. Any spattering or spraying of the sewage caused by high speeds or improper design, especially in the application of air, steam, or water jets, for cleaning, increases the odor.

Another and more probable cause for offense lies in the disposal of the screenings. From observation of a great many plants, the writer is convinced that screenings should either be incinerated, artificially dried, or promptly removed from the works. During warm weather, this should not be delayed more than 24 hours from the time of collection. Offensive conditions may be mitigated by composting with sweepings. This question of disposal is one of much importance, and also one which is frequently neglected.

Accessibility.—All parts of a screen should be accessible, for cleaning, repair, and renewal, with the least interruption to operation.

The Hamburg screen is hung on a shaft so that the lower end may be swung above the surface when required. The rotation of the wing, shovel-vane, and drum screens renders all moving parts accessible, and, with the exception of the bearing at the foot of the shaft, this is true of the Riensch-Wurl screen. It is understood that this objection is to be overcome by suspending the screen from a bearing above. In the case of the Frankfort screen, nearly all the moving parts can be replaced without interruption of the flow.

First Cost.—The cost of a screening plant depends, not only on the screen with its auxiliary machinery, but, to a great extent, on the location, foundations, and size of chamber. Simplicity of design, compactness, and the loss of head resulting from its use are all to be considered; in fact, the last may control the selection. As the cost depends on so many factors, the figures of gross cost, which are usually the only ones published, are of little value in designing new plants. Relative costs cannot be given for the several types, but approximate cost figures for Riensch-Wurl screens in Germany, as given to the writer by Mr. Wilhelm Wurl at the factory at Weissensee, are stated in Table 12. They show that the price increases with the size and capacity, and that the cost per million gallons capacity decreases.

With the drum or shovel-vane screen, it is possible to do away with the cost of a grit chamber, as these screens can handle this material as well as the screenings. The wear of combined sewage with its contained grit on a fine-mesh screen would make the removal of grit before screening desirable where these conditions obtain.

Cost of Operation.—The costs of operation depend primarily on the power consumed, supplies, and attendance; but they also depend on depreciation, repairs, and renewals. The materials used should be

TABLE 12.—COST OF RIENSCH-WURL SCREENS.

Diameter.	Normal capacity, in cubic feet per second.	PRICE.	
		Per screen.	Per million gallons per day.
4 ft. 3 in.	0.6	$720	$1 850
5 ft. 0 in.	0.9	840	1 440
6 ft. 6 in.	......	1 200	....
9 ft. 10 in.	......	2 400	....
14 ft. 9 in.	14	3 600	397
16 ft. 6 in.	25	4 800	297
19 ft. 9 in.	35	6 000	266
23 ft. 0 in.	70	8 400	185
26 ft. 4 in.	160	12 000	116

TABLE 13.—COST OF OPERATION OF SCREENS AT MAINZ, WITH DIFFERENT RATES OF FLOW.

RATE OF FLOW:		COST:	
In cubic feet per second.	In liters per second.	In dollars per million gallons.	In pfennigs per cubic meter.
2.1	60	$1.76	0.195
2.8	80	1.32	0.145
4.9	140	0.74	0.082
6.3	180	0.58	0.064
8.4	240	0.43½	0.048
11.2	320	0.32½	0.036

durable, and subject to the least possible corrosion. The Windschild drum screen involves the expense of compressed air, but this objection may be counterbalanced by the decreased moisture in the screenings. The Riensch-Wurl and shovel-vane screens involve the periodical renewal of brushes. The latter type is not simple in design, and for this reason might cost more for repairs, but is believed to be well made. The Hamburg screen is subject to the wear on many moving parts. The Weand screen has shown indications of deterioration from corrosion and wear. Experience with these screens has been too brief to assign to each a probable period of life. One can only form an opinion in each case from its design and the conditions under which it is to operate.

FINE-SCREENING VERSUS SEDIMENTATION.

It has been shown by the data already presented that in the matter of efficiency the best fine-screening will compare well with such tank treatment as may usually be looked for. That is, from 30 to 50%

of the suspended solids may be removed by fine screens, as compared with 50 to 65% by sedimentation.

Emscher tanks have a particular advantage in producing a sludge which is inoffensive, even if stored for a considerable period, and may be used for filling in waste land or, possibly, as a filler for fertilizer.*

Tank treatment uses up little head and is subject to little deterioration, such as is inevitable with machinery of any kind, and although the cost of attendance may be a little greater, such attendance, except for general supervision, may be of a somewhat lower grade than is necessary when motors, screens, conveyors, and possibly boilers, engines, or air compressors have to be looked after.

Monti, after the Berlin experiments already referred to, concluded: "that although this fine screening removes all of the offensive-looking matter, yet the liquid remains quite turbid, and that much better results can be obtained by a few hours' sedimentation in large tanks."†

This is confirmed by G. M. Wisner, M. Am. Soc. C. E., who concludes, from experiments at Chicago,‡ that "there is little or no improvement in the stability of the local sewage due to screening through a device with 40 meshes to the inch."

On the other hand, tank treatment requires considerable area, and this is sometimes difficult to secure at moderate cost. It would be a great expense, for instance, in some parts of New York City. Then, with plain sedimentation, a large volume of watery sludge, which becomes very offensive if stored, must be disposed of, and this means an additional area for drying, with, perhaps, additional cost for artificial drying or for transportation to sea while in a crude condition. Emscher tanks require less land than other forms, but are more costly to construct, their great advantage being the favorable quality of the sludge.

The following extracts from a report by George W. Fuller, M. Am. Soc. C. E., to the Metropolitan Sewerage Commission of New York,§ are of especial interest, as coming from one who can speak with authority on these matters. He says:

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* "Fresh and Decomposed Sludge," L. C. Frank, *Engineering Record*, Vol. 68, p. 331.

† E. Kuichling, *Proceedings*, Thirty-second Annual Meeting, New Jersey Sanitary Assoc., 1906.

‡ "Sewage Disposal," by George W. Fuller, M. Am. Soc. C. E., p. 381.

§ Report of April 30th, 1914, pp. 209-210.

"Fine screens afford the cheapest way of removing visible objects of sewage origin from the waters receiving sewage where such screening treatment alone is sufficient for obtaining satisfactory results. Under conditions where the limit is at times reached in the amount of clarified sewage which a watercourse will oxidize satisfactorily, settling tanks as a general rule are cheaper to install than screens, because for a given cost they will remove a greater quantity of organic matter.

* * * * * * * *

"In my opinion, screens are preferable to settling tanks only where it is desirable or necessary to remove only relatively large sewage matters in suspension. Where settling solids would form deposits in the watercourses if screening alone were adopted, to install settling tanks will prove wiser than to install fine screens."

Screening plants, however, are very compact and, with grit chambers, need never occupy more than a fraction of a city block. The process is cleanly and largely automatic, requiring very little attendance. The product is much more readily handled, as it contains less moisture, as a rule, than sludge—from 75 to 85% instead of 90 per cent. It is true that some screenings may contain as much moisture as well-digested Emscher sludge, or perhaps more, but it is not, as in the latter case, in a liquid condition, but may be transported in ordinary wagons or cars after a brief period of draining.

If a recovery of grease or fertilizing value is considered, the dried screenings will be found, as a rule, to contain more organic matter than the dried sludge, and, therefore, pound for pound, to yield a greater return. Unless the plant is of considerable size, the screenings can best be drained and then burned under boilers or in a special incinerator, plans for which, it is understood, are now under consideration by a suburb of New York City.

To sum up, although screens may remove less material than tanks, the difference is not very great in the best types of each, and still less when one considers the organic constituents, the presence of which is most objectionable in an effluent. The land required is much less than for tanks, and the cost of installation is probably less, although this depends on local conditions. As to gross annual charges, no general comparison is possible, but it is believed that screens will often be found quite as economical as tanks if the fixed charges are included and the total dried organic matter removed is used as a basis of estimate, in spite of the fact

that there is greater deterioration and that higher rates of wages may be required with screening.

The head lost in either case need not be great, except in the original design of the Weand screen, or where, as with the Carshalton screen, it is used to develop power for operation.

CONCLUSION.

The points which the writer has attempted to bring out may be summarized as follows:

The use of fine screens for sewage deserves more serious consideration than it has received heretofore in America. Although tank treatment will continue to be the standard preliminary process, under most conditions, from economic considerations, there are now several screens which will remove approximately as much of the organic suspended matter from sewage while occupying much less space and delivering a product that is both richer in manurial ingredients and more readily handled than tank sludge.

For the foregoing reasons, it is the writer's opinion that fine-screening will be adopted in the future by many towns situated on bodies of water which are capable of assimilating the effluent; possibly, also, as a preliminary process to tank treatment, filtration, and disinfection. On the other hand, the relatively high cost of attendance and the probable lack of a market for the screenings, when compared with conditions abroad, will probably serve to prevent the marked increase in their use in the United States which has been experienced in Germany.

In view of a reasonable development of fine-screening in America, more fundamental data as to efficiencies and costs are greatly to be desired. These are rarely available in their best form, even in Germany.

The volume of sewage received at the plant, or the population served, should be recorded, and also either (*a*) the suspended solids and total organic matter, or (*b*) the parts per million of average suspended solids in the crude sewage, the weight of the dried grit and screenings, and the percentage of each which is organic.

a.—The suspended solids and total organic matter in the crude sewage and after each operation should be recorded for the different times of the day and week, and then, if weighted according to the

flow when sampled, it will furnish the quantity of these essential materials that have been removed by the screens and such preliminary devices as coarse screens and grit chambers.

b.—A record should be kept of the parts per million of average suspended solids in the crude sewage, the weight of the dried grit and screenings, and the percentage of each which is organic.

Data of this kind from different sources would then be comparable and of much value in planning new works. The costs, too, not only of the screens, but of foundations, auxiliary machinery, and housing, with those for supplies and attendance, should be matters of record. So much stress laid on these things would perhaps be unnecessary, but for the fact that such essential data are, with few exceptions, not to be had.

Finally, the adoption of screens should be considered:

1. Where the principal requirement is the removal of the larger solid matters;
2. Where land values are high;
3. Where the cost of excavation for tanks is high;
4. Where the cost of sludge disposal is high, or where such disposal is likely to cause objectionable odors; and
5. Where the recovery of grease and fertilizer is an important factor.

With openings not more than 0.10 in. (2½ mm.) in size, fine-screening should remove at least 30% of the suspended solids and 20% of the suspended organic solids from ordinary domestic sewage, or 0.1 cu. yd. of screenings, containing 75% of water, per 1 000 population daily.

The writer expresses his sincere desire that the many members of this Society who have studied the subject of fine-screening, will present such information and as many suggestions relative to the matter as possible.

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DISCUSSION

Mr. Kershaw.

G. Bertram de B. Kershaw,* M. Am. Soc. C. E. (by letter).—The first point noted, in reading Mr. Allen's valuable paper, is that it deals almost exclusively with large cities and their sewerage installations.

It is suggested in the paper that one of the conditions where fine-screening should generally be considered is "where the removal of all but the very fine solids will furnish a satisfactory effluent." It follows from this that the city contemplating fine-screening as a final process must have a suitable and sufficient body of diluting water available to carry out the purification process proper by oxidation, effected by abstraction of dissolved oxygen from the diluting body.

Conditions in England (where almost all the writer's first-hand experience has been obtained), are, as a general rule, unfavorable for the adoption of processes which throw the brunt of the purification on the body of water into which the sewage liquor is discharged.

English rivers, with very few exceptions, are small, in marked contrast to many of the rivers of the Continent; moreover, many of the smaller rivers receive sewage effluents at short intervals of distance throughout almost their entire courses, and are not in a condition to cope with any fresh burden of self-purification.

It is found to be the case, therefore, that screening is almost entirely confined to what may be termed "coarse" screening, that is, a screening process only intended to remove the grosser solids, such as sticks, stones, coal, corks, potato peelings, and the like, which otherwise would be likely to lodge under sluice-valves and penstocks, and cause trouble; screens are also provided when the sewage is subsequently pumped, or, to protect the filter presses, where sludge pressing is carried out.

Such screens are generally of the bar type, cleaned by hand or automatic rakes, the openings between the bars being from about ½ in. to 1 in., and greater.

As giving a good idea of a very usual type of rough screen used at many sewage works in England, the following particulars regarding the Stratford-on-Avon sewage works (as being typical of a small English country town) may be of interest.

The screen at the pumping station is of ⅜-in. bars, spaced ⅜ in. apart, and it is inclined at an angle of about 20° with the perpendicular. Cleaning is effected by a system of traveling rakes attached to two endless chains, which elevate the screenings and tip them into a tumbril cart. It should be observed that the Stratford sewage is a strong one and contains a good deal of brewery waste.

* London, England.

Mr. Kershaw.

Table 14 gives particulars relating to the screening operations.

TABLE 14.—ANNUAL QUANTITY OF SCREENINGS PRODUCED AT STRATFORD-ON-AVON SEWAGE WORKS.

Year.	Total yearly sewage flow, in millions of gallons.	Tons of screenings.	Tons, per million gallons.	Cost of Removing Screenings.		Rainfall for the year, in inches.
				Per million gallons.	Per long ton.	
1908-09	126	100.5	0.70	1s. 3d.	1s. 7d.	21.06
1909-10	134	103.5	0.77	1s. 1d.	1s. 3½d.	26.95
1910-11	136	99.0	0.73	5.9d.	8.1d.	25.17

Where special trade sewages have to be dealt with, such as those containing wastes from wool combing or the cotton industry, screening forms an important integral part of the sewage plant, but it should be observed that such screening is only to be regarded as ancillary or preliminary to the purification process proper, and it is intended merely to remove substances which might cause mechanical troubles at a later stage, reliance being placed on subsequent processes for real purification. It is, in fact, a process of abstraction rather than of purification, as generally understood.

The writer believes he is correct in saying that there are very few, if any, screening plants in England which are designed to effect other than a partial withdrawal of the larger suspended solids, such as are peculiarly likely to float and give an unpleasant appearance to the water into which they happen to be discharged. Certainly there are no fine-screening plants capable of producing the results mentioned by Mr. Allen as being achieved in Germany by the best examples of fine screens.

Fine-screening would appear to be peculiarly adapted to seaside resorts in England, where bathing is enjoyed.

The Royal Commission on Sewage Disposal,* for which the writer has been Engineer for some sixteen years, in dealing with standards, states as follows:

"But we think it of great importance, if only for the sake of the appearance of the river, that the grosser solids, such as undisintegrated fæces, paper, etc., should, as far as possible, be held back, and the provision of some effective form of screening apparatus should be insisted on wherever practicable."

There is no doubt that, when the treatment of sewage by dilution is better understood in England, fine-screening will become of considerable importance. Screening as a final process in certain cases is likely to be carried out when the Commission's reports have been digested by

* Eighth Report, Royal Commission on Sewage Disposal, p. 12, para. 42.

Mr. Kershaw.

the English Local Government Board. At present, there is some uncertainty as to how "dilution" schemes would be received officially.

The following statement shows the degree of dilution and limiting figures for suspended solids in sewage liquors, laid down by the Commission for cases where disposal by dilution is to be countenanced.

Dilution.	Suspended solids, in parts per 100 000.
Exceeding 150 volumes and less than 300 volumes.......	6
Exceeding 300 volumes and less than 500 volumes.......	15
Exceeding 500 volumes........	Crude sewage could be discharged subject to such conditions, as to the provision of screens or detritus tanks, as might appear necessary to the central authority.

It may be noted that the following figures show what would be considered good tank liquors in England, assuming a domestic sewage of average strength, containing some 35 parts of suspended solids per 100 000, to have been treated.

Treatment.	Parts per 100 000.
Septic tank treatment...............	10 to 15
Continuous flow settlement..........	10 to 15
Quiescent settlement	5 to 8
Continuous flow precipitation........	3 to 6
Quiescent precipitation	1 to 4

It should be observed that, for the determination of the suspended solids, a Gooch crucible, packed with asbestos floss, is used.

An English domestic sewage of average strength contains about 35 parts of suspended matter per 100 000, but the quantity is variable. A large part of these solids is usually very finely divided. The actual fineness will depend to a great extent on the sewerage system, the distance the sewage has traveled, whether pumped (which often breaks up the solids and increases colloidal matter) or flowing by gravitation, nature of street surfaces, if the combined system of sewers is in use, temperature, and many other variable factors; but a large proportion of the finer suspended solids will almost invariably pass with ease through a mesh of $\frac{1}{50}$ in. or even less; and, with a septic sewage, such as sometimes results in hot weather when the sewage has traveled far, the finest screens will effect very little useful work.

In most sewages, trade wastes of various kinds form a feature, and the suspended solids in the sewage may then be very greatly increased, and vary from hour to hour. Again, a population served

entirely by water-closets will, as a rule, contribute much more screenable matter than where the whole of the fæces and paper does not enter the sewers. Mr. Kershaw.

The writer has followed the Chicago sewerage problem for some time past, and the percentage of removal effected by Mr. Jennings' screen (63%) seems very high, notwithstanding the fineness of the mesh (40 per in.). Possibly it is accounted for by the nature of the sewage, which presumably contains suspended matter of a uniform nature and size, such as masticated hay, etc. If this is so, such screenings would be likely to resist decay, owing to the large quantity of cellulose matter they would contain.

The nature of the suspended solids in a sewage varies considerably. From 40 to 60% of the suspended matter in most English sewages consists of mineral or inorganic matter, practically innocuous by itself (excepting as regards the silting up of waterways), but it is found that these particles carry with them, as a kind of envelope, a considerable quantity of organic impurity in solution, and they exercise a de-oxygenating action on rivers and streams, albeit not such a sustained action as solids of organic origin.

When dilution is relied on to complete the purification of a settled or screened sewage liquor, the main essential is to keep the bulk of the suspended matter in suspension in the river water until it has become oxidized or nearly so.

The capacity of a clean river to endure pollution without visible signs of nuisance appears to be closely related, among other things, to its depth, velocity, power of transportation, temperature, and aquatic plant, fish, and insect life.

If the mud near the borders of a river receiving a certain quantity of pollution be examined, it will generally be seen to consist of very fine particles of black mud, overlaid by a thin layer of brown (or nearly oxidized) mud on the surface, where it has been intimately in contact with the dissolved oxygen in the river water. How fine much of this mud is, may be judged by mixing some of it with tap water and straining the liquid through a handkerchief into a glass; much of the fine matter will be found to pass through the linen with the water.

It is mainly these very fine solids which cause de-oxygenation troubles in sluggish rivers or sluggish reaches of rivers, when they become deposited, and it is very difficult, with good tank treatment, to keep them back, unless precipitants are used.

In England, however, there are instances where, owing to favorable local conditions in a river—such as volume and velocity—such fine suspended matter might be discharged without any perceptible damage to the river, and in such cases fine-screening of the nature indicated by Mr. Allen would be of marked value.

Mr. Kershaw.

The writer has seen examples where the removal of the tank effluent outfall from the bank of a river to a point some distance from the shore would have admitted of fine-screening in lieu of tank treatment. It is quite possible to foul a strip of river seriously by the discharge of tank liquor or sewage close to the shore, whereas the admixture of screened sewage with the whole volume of river water would suffice for purification purposes.

With small rivers in England, the writer has no doubt that less damage would result to the rivers by passing fresh sewage from a small community into them, after efficient screening, than, as is often done, by passing the sewage through a septic tank and thence to the river. Septic tank treatment almost invariably results in the production of sulphuretted hydrogen, which has the property of taking up dissolved oxygen from water very rapidly. An Imhoff tank would probably solve such problems, but screening would be cheaper.

As Mr. Allen carefully points out, screenings consist largely of organic matter, and considerable weight must be attached to the fact that, with suitable forms of screens, organic matter, such as fæces, etc., are removed at once from contact with the liquid portion of the sewage, whereas, with tank treatment, much of this organic matter is left in contact with the liquid in the tank for some considerable time, more and more of it going into solution as fermentation proceeds.

On the other hand, screenings are more likely to attract dipterous insects for ovipositing than sludge; still, they might be deterred by the use of bleaching powder or some similar chemical. Both screens and screenings should be roofed in wherever possible, for the foregoing reason, quite apart from danger of nuisance from smell.

The results obtained by fine-screening in Germany, cited by Mr. Allen, appear to show an extraordinary efficiency, and it is difficult to imagine such a percentage as 90.

Experiments carried out at Leeds, England, with a fixed-mesh screen of 30 per in., inclined with the sewage flow, appeared to keep back a considerable proportion of the suspended matters in the sewage, but "although the accumulation seemed bulky, it was found on drying to represent less than 10% of the suspended matter in the sewage." (Leeds sewage contains about 60 parts of suspended matter per 100 000.) It may be noted that the clear liquid of a sample of average Leeds sewage which had been passed through filter paper still contained in solution about 50% of the original impurities, becoming dark-colored and putrefying.

There can be no question that screenings and sludge stand on an entirely different footing as regards manurial value and fat content. Screenings consist largely of nitrogenous matter, and usually contain

little mineral matter; whereas, in sludge the solids are generally 50% inorganic, and it is worth just what can be obtained for it. Mr. Kershaw.

The fat content of the screenings at Hamburg is well brought out in Table 1, showing the composition of grit and screenings, the average being nearly 5 per cent. In Table 2 it would be interesting to know whether the nitrogen and phosphates are in as readily available form for plant life as in artificial manures. The manurial constituents of most sewage sludges appear to be in an inert and slowly-acting condition, and the writer has always considered that a great part of any of the value that sewage sludge may possess, is due to its physical and mechanical, rather than its manurial, action on soils.

Regarding the Carshalton screen, it should be observed that all these screens are used for either crude or grit-settled sewages. The writer does not know of any plants in which they are used for screening tank liquors. It may be noted that this screen is driven by a water-wheel actuated by the flow of sewage, and the speed at which the screen travels is automatically adjusted to the work required of it.

This is a matter of importance, because, if a screening apparatus reduces the velocity of the sewage flow beyond a certain point, settlement of suspended matter will take place up stream from the screen; it is of importance, therefore, for automatic screens to work at a rate which will allow the sewage flow to be maintained at a velocity sufficiently high to keep the solids in suspension. If, owing to too slow a motion of the screen, or temporary insufficient area of mesh openings (such as might result from paper flattening against the meshes), the decrease in velocity causes the sewage to head up, and, consequently, adjacent storm overflows may be brought into operation, especially if the gradient of the main outfall sewer is a slack one.

It is a most complicated matter to arrive at the real efficiency of screens, and this is largely due to the difficulty of obtaining reliable average samples, and of procuring accurate analytical figures covering a long period of experimentation.

The accurate determination of suspended solids is a tedious process, and it might be possible to use a centrifuge in order to save time. If this was done, however, results obtained with a sewage containing suspended solids of a granular nature would scarcely be comparable with those obtained elsewhere with a sewage containing flocculent suspended solids, although they would be reliable for the one place if the suspended solids were constant in character.

The most important point appears to the writer to be, not how great a quantity of screenings is removed per million gallons of sewage, but how much suspended matter is left in the screened liquor. A screen may appear to remove a large quantity of matter from the sewage (as in the Leeds experiments), and yet, on carefully analyzing

Mr. Kershaw. the screened liquor, the quantity of suspended solids is usually found to be high.

It is of vital importance, therefore—and it is noted that Mr. Allen lays stress on this point—that, in any screening experiments, the quantity of suspended solids, both in the raw sewage and in the screened liquor, should be most accurately determined, and for a considerable period, during both hot and cold weather. During cold weather one would expect more screenings to be removed than in hot weather, and, turning to the Dresden results, given in Table 10, if the weight of dry material in the screened liquid be subtracted from the weight of dry material, as in Table 15, it will be seen that the matters removed by screening reached a maximum in the coldest weather, and this is what one would have expected, because fermentation and its power of breaking down organic matter is then at a low ebb.

TABLE 15.

Period.	Weight of dry material in raw sewage, in grammes.		Weight of dry material in screened liquid, in grammes.	Weight of dry material removed by screens, in grammes.
March, April, May..................	50.96	less	41.40	= 9.56
June, July, August.................	61.42	"	50.78	= 10.64
September, October, November..	81.71	"	59.18	= 22.58
December, January, February...	99.41	"	51.33	= 48.08

It is essential, in any comparative trials of screens, that all samples should be average ones, that is, drawn every hour or oftener throughout the 24 hours, according to the rate of flow of the sewage, otherwise very misleading results may follow; further, the samples of raw sewage and of screened sewage should be strictly corresponding ones; these can be ensured as a rule by the use of a coloring dye, such as fluorescin (green) or eosin (red).

In connection with this last point, the figures of analysis given in Table 4 for chlorine, show that the samples were not corresponding ones, the figure for the raw sewage being 262.6, and that for the screened sewage, 279.4; these figures should have been practically identical. It is to be noticed that there is no material difference in the polluting property of the raw and screened sewages, judged by the figures for "oxygen consumed."

Regarding the results given in Table 4 with reference to the Geiger screen, it appears probable that the rolling action of the vanes, as the screenings pass to the axis, tends to comminute, or perhaps to send into solution, some organic matter, thereby increasing the strength of the screened sewage.

Mr. Kershaw.

Some methods of cleaning screens leave much to be desired, from this point of view, and sewages are sometimes stronger as regards dissolved impurities after screening than before, from this cause. Good management must always be forthcoming, if the best results are to be obtained.

In order to gain a strictly scientific comparison between various screens, it would seem that the only way would be to test them all on the same sewage; even this procedure might not hold good, because a particular type of screen might be better adapted to some sewages than to others.

In any case, trials of screen efficiency carried out with one sewage would be costly, because they would need to be tried on a working scale to yield useful results.

A point needing special attention would be the moisture contained in the screenings, and it would seem advisable to devise a standard method of draining, whereby the screenings in each test would be drained for a given time on a specially formed drying area before being sampled for analysis; moreover, the sub-sampling and sampling of the screenings would require very great care.

The hourly sewage flow and the velocity through the screens would need to be carefully recorded in all trials, because solids are apt to be forced through even fine screens under conditions of heavy sewage flow.

Table 11, summarizing the data relating to the screens discussed by Mr. Allen, is valuable, and the cost of operation does not anywhere appear to be excessive; it is to be observed, however, that the whole question of efficiency depends on the sampling and analysis, and the accuracy with which these are done.

Will Mr. Allen state whether the figures in Table 11 for cost of operation per million gallons include interest on capital outlay, or depreciation? The writer is of the opinion that they do not.

It would have been most interesting if the author could have given columns showing the volumes of water into which the screened liquors were discharged, and the visual effect produced on the water and banks. The writer is well aware, however, of the difficulties involved in obtaining reliable information on these points.

The writer is in general agreement with Mr. Allen's conclusion, although rather doubtful whether a 30% removal of total suspended solids from an average domestic sewage is not too sanguine an estimate to be uniformly attained.

Mr. Allen is to be congratulated on a most interesting and instructive paper. A vast amount of time and labor must have been expended in collecting and assorting the minute data it contains, and it would be well if more engineers in England would follow Mr. Allen's example

Mr. Kershaw. and go and see for themselves anything new connected with sewage purification and disposal plants.

A feature of this paper is the excellence of the half-tones with which it is illustrated.

Mr. Stevenson. W. L. STEVENSON,* ASSOC. M. AM. SOC. C. E. (by letter).—During 1909-10, the Bureau of Surveys of Philadelphia operated a sewage testing station in connection with the investigations being carried on for the preparation of a comprehensive plan for the collection, treatment, and disposal of the sewage of that city.

Among the processes studied was fine-mesh screening. The observations throw light on the second and third functions of fine screens mentioned by Mr. Allen, and also on the character of the sludge produced in subsequent sedimentation, and the efficiency of fine-screening as a preparatory treatment for subsequent disinfection.

The sewage used in the testing station was collected by a separate system of sewers from a territory partly residential and partly devoted to the textile industry, and contained a considerable quantity of trade wastes.

The fine-mesh screen consisted of a cone covered with red metal cloth, 32 meshes per inch, and having clear openings ½ mm. square. Sewage which had been removed from the side at about mid-depth of the flow in the intercepting sewer was raised to the testing station by a plunger pump and applied to the screen through twenty-four ¼-in. nozzles. No attempt was made to design a screen for practical use, the screenings being washed away to a trough by the splashing of the jets; but screened sewage was obtained for subsequent study. On an average, the sewage applied to the screen contained 200 parts per million of suspended matter, as measured by the Gooch crucible; and the average effluent contained 133 parts per million. The functions of this screen on subsequent treatment of the effluent were as follows.

"2. The Removal of Solids That Otherwise Would Cause the Frequent Subsequent Clogging of Sprinkler Nozzles or Filters."—The screened sewage, after plain sedimentation in a horizontal tank, was applied to a percolating filter through two square spray Taylor nozzles. During 9½ months service, these nozzles were never clogged; whereas, the same crude sewage after similar sedimentation without fine-screening caused frequent nozzle-clogging from match sticks, wool fibers, etc. The interstices of the media in the filter to which the screened and settled sewage was applied, were invariably cleaner and freer from sludge than those in filters to which only settled crude sewage was applied.

"3. The Removal of Material Likely to Promote the Formation of Scum in Subsequent Tank Treatment."—Two horizontal-flow, sedi-

* Philadelphia, Pa.

mentation tanks, each provided with baffle- and scum-boards of identical construction and operated at the same rates, received as influents, respectively, crude sewage and the effluent of the fine screen. In the tank receiving crude sewage a scum always formed a few days after it was put in service, and before cleaning the tank it usually attained a thickness of more than 1 ft. It was composed of a leathery mat of wool fibers, hops, and other sewage matters which were light enough to float. Mr. Stevenson.

The surface of the tank receiving the screened sewage always remained clean and free from any floating matters. This had an effect on the quality of the effluent. Septic action developed much sooner in the tank receiving unscreened sewage, and the effluent was more deoxidized than that from the tank receiving screened sewage.

Naturally, the effluent of the tank receiving screened sewage was lower in suspended solids than the other, and its quality was also more uniform. Sudden increases in suspended solids in the crude sewage produced a synchronous increase in those in the effluent of the tank receiving crude sewage; but no change was noted in the effluent of the tank receiving screened sewage, as the screen relieved the tank from these peak loads of suspended matter.

The Effect of Fine Screening on Sludge Formed in Subsequent Tank Treatment.—On an average, 4.07 cu. yd. of sludge of 1.053 specific gravity and containing 86.1% of moisture, were deposited from each 1 000 000 gal. of crude sewage settled, and 4.65 cu. yd. of sludge of 1.036 specific gravity and containing 90% of moisture, were deposited from each 1 000 000 gal. of screened sewage settled. The lower specific gravity and higher water content of the sludge from the screened sewage was due to the fineness of the particles of which it was composed. Each cubic yard of the crude sewage sludge contained, on the average, about 990 lb. of dry residue and 790 lb. of the screened sewage sludge.

The effect of the fine screen, therefore, was to increase the volume of sludge by 14%, but to reduce the quantity of dry solids deposited by 20 per cent.

The screened sludge, however, dried more readily than the crude sludge, due to absence of wool fibers which prevented the formation of deep cracks in the latter. When removed from the drying beds, the quantity of screened sewage sludge was less than that of crude sewage sludge.

Efficiency of Fine Screening as a Preparatory Process to Disinfection.—Experiments were carried on to determine the quantity of calcium hypochlorite required to disinfect fine-screened sewage, settled crude sewage, and settled screened sewage. The results were divided into two groups: First, a quantity of disinfectant which should be added during an epidemic of intestinal disease to

Mr. Stevenson. insure protection to communities below the point of discharge; and, second, a quantity which would destroy more than 95% of the *B. Coli* at a moderate cost. The results are given in Table 16,

TABLE 16.

Kind of sewage.	Available chlorine added, in parts per million.	Total Number of Bacteria per Cubic Centimeter, on Gelatine at 20° Cent., in 48 Hours.			*B. Coli* per Cubic Centimeter, as per Jackson's Presumptive Test.		
		Initial.	Final.	Percentage removed.	Initial.	Final.	Percentage removed.
Fine-screened sewage....	12.4	2 470 000	337	99.99	121 000	20	99.98
	6.0	2 060 000	181 000	91.21	149 000	7 470	95.42
Settled crude sewage....	11.9	2 450 000	350	99.99	143 000	10	99.99
	5.4	760 000	31 000	95.92	67 000	745	98.89
Settled screened sewage.	12.0	2 130 000	310	99.99	86 000	20	99.98
	4.3	660 000	22 500	96.59	317 000	1 350	99.57

and show that sedimentation of the sewage was more efficient as a preparatory process to disinfection than fine-screening; but that, in cases where the receiving body of water is capable of assimilating, without nuisance or detriment, the effluent of a fine-mesh screen containing particles not larger than 1 mm., it is practical to disinfect it. Experiments have been made to show that it is not practical to disinfect sewage containing particles larger than 1 mm.

Summary.—The results of the experiments made by the Bureau of Surveys show that the use of a screen of 32 meshes per inch prior to sedimentation produces an effluent which will not clog the nozzles or media of a percolative filter; prevents the formation of scum in subsequent tank treatment; causes a uniform quality of tank effluent, regardless of irregular quantities of suspended matter in the crude sewage; yields a sludge from subsequent sedimentation higher in moisture and of greater volume, but containing less dry solids, than from the sedimentation of crude sewage; and is a satisfactory preliminary treatment for subsequent disinfection.

These data are merely statements of facts. The application of fine-screening of sewage to any particular case is an engineering problem to be determined by the local needs and conditions.

The writer wishes to express his appreciation of the valuable data which Mr. Allen has furnished in this paper.

Mr. Whipple. George C. Whipple,* M. Am. Soc. C. E. (by letter).—This is an excellent paper on sewage screening, and should form the nucleus for some interesting discussions. With the author's main contention, that fine-screening is destined to play an important part in sewage

* Cambridge, Mass.

treatment in America, the writer is in hearty sympathy. That was the conclusion which he reached a number of years ago after a trip to Europe which was referred to in a discussion presented to the Society in June, 1906. Mr. Whipple.

In the prevention of the pollution of streams, lakes, and harbors it is necessary to have laws which govern the discharge of sewage and trade wastes into such bodies of water. These laws must not be discriminatory, but, in the interest of fairness, must be general in their application, and yet each city or town must be permitted to take charge of its local facilities for sewage discharge, using them to the limit, but with due regard to proper sanitation. Undoubtedly, there are local situations where the discharge of raw sewage into streams will do no harm, but it is difficult to frame and execute laws which will fairly separate the places where no sewage treatment is needed from those where treatment of some kind is necessary. From an administrative point of view, it may be better to make the law general that no raw sewage shall be discharged into our waterways, and leave the nature of the treatment in each case to some properly constituted authority. In doing this it will be necessary to give a broad meaning to the term "sewage treatment" and allow it to cover, not only the more elaborate processes of purification, but also the simpler methods of coarse screening.

Between raw sewage and that which has passed through coarse screens there is no material bacteriological difference, and we might almost say the same with reference to raw sewage and that which has passed through the fine screens described by the author. Yet the process of screening is so inexpensive, and it does so much to avoid the obvious signs of pollution by the removal of the lighter and the grosser solids, that there are few cases where the process is not justified by its results. By giving this liberal interpretation to the expression "sewage treatment" it will be easier to obtain legislative action relating to the control of sewage discharge. The engineers who recently advised the International Joint Commission, in the matter of the control of sewage disposal for communities on the boundary waters between the United States and Canada, took this view when they formulated the following guiding principles. It will be seen that they were worded so as to include screening as a form of treatment. For this reason they may be appropriately inserted in this discussion.

"New York, June 27, 1914.

"To the International Joint Commission.

"Gentlemen: The following statements represent the essence of the opinions given by us before your honorable commission at the conference held in New York on May 26 and 27, 1914:

"1. Speaking generally, water supplies taken from streams and lakes which receive the drainage of agricultural and grazing lands,

Mr. Whipple. rural communities, and unsewered towns are unsafe for use without purification, but are safe for use if purified.

"2. Water supplies taken from streams and lakes into which the sewage of cities and towns is directly discharged are safe for use after purification, provided that the load upon the purifying mechanism is not too great and that a sufficient factor of safety is maintained, and, further, provided that the plant is properly operated.

"3. As, in general, the boundary waters in their natural state are relatively clear and contain but little organic matter, the best index of pollution now available for the purpose of ascertaining whether a water-purification plant is overloaded is the number of *B. coli* per 100 cubic centimeters of water expressed as an annual average and determined from a considerable number of confirmatory tests regularly made throughout the year.

"4. While present information does not permit a definite limit of safe loading of a water-purification plant to be established, it is our judgment that this limit is exceeded if the annual average number of *B. coli* in the water delivered to the plant is higher than about 500 per 100 cubic centimeters, or if in 0.1 cubic centimeter samples of the water *B. coli* is found 50 per cent. of the time. With such a limit the number of *B. coli* would be less than the figure given during a part of the year and would be exceeded during some periods.

"5. In waterways where some pollution is inevitable and where the ratio of the volume of water to the volume of sewage is so large that no local nuisance can result, it is our judgment that the method of sewage disposal by dilution represents a natural resource and that the utilization of this resource is justifiable for economic reasons, provided that an unreasonable burden or responsibility is not placed upon any water-purification plant and that no menace to the public health is occasioned thereby.

"6. While realizing that in certain cases the discharge of crude sewage into the boundary waters may be without danger, it is our judgment that effective sanitary administration requires the adoption of the general policy that no untreated sewage from cities or towns shall be discharged into the boundary waters.

"7. The nature of the sewage treatment required should vary according to the local conditions, each community being permitted to take advantage of its situation with respect to local conditions and its remoteness from other communities, with the intent that the cost of sewage treatment may be kept reasonably low.

"8. In general, the simplest allowable method of sewage treatment, such as would be suitable for small communities remote from other communities, should be the removal of the larger suspended solids by screening through a one-fourth inch mesh or by sedimentation.

"9. In general, no more elaborate method of sewage treatment should be required than the removal of the suspended solids by fine screening or by sedimentation, or both, followed by chemical disinfection or sterilization of the clarified sewage. Except in the case of some of the smaller streams on the boundary, it is our judgment that such oxidizing processes as intermittent sand filtration, and treatment by sprinkling filters, contact beds, and the like, are unnecessary,

inasmuch as ample dilution in the lakes and large streams will provide sufficient oxygen for the ultimate destruction of the organic matter. Mr. Whipple.

"10. Disinfection or sterilization of the sewage of a community should be required wherever there is danger of the boundary waters being so polluted that the load on any water-purification plant becomes greater than the limit above mentioned.

"11. It is our opinion that, in general, protection of public water supplies is more economically secured by water purification at the intake than by sewage purification at the sewer outlet, but that under some conditions both water purification and sewage treatment may be necessary.

"12. The bacteriological tests which have been made in large numbers under the direction of the International Joint Commission indicate that in most places the pollution of the boundary waters is such as to be a general menace to the public health should the water be used without purification as sources of public water supply or should they be used for drinking purposes by persons traveling in boats.

"13. It is our judgment that the drinking water used on vessels traversing boundary waters should not be taken indiscriminately from the waters traversed, unless subjected to adequate purification, but should be obtained preferably from safe sources of supply at the terminals.

"14. While recognizing that the direct discharge of fecal matter from boats into the boundary waters may often be without danger, yet in the interest of effective sanitary administration it is our judgment that the indiscriminate discharge of unsterilized fecal matter from vessels into the boundary waters should not be permitted.

"Yours, respectfully,

"George W. Fuller.
Earle B. Phelps.
George C. Whipple.
W. S. Lea.
T. J. Lafreniere."

Mr. F. A. Dallyn, who was one of the six consulting engineers to advise the Commission, presented a minority report, in which he eliminated Paragraphs 5, 7, and 11, and made a few slight changes in some of the other paragraphs.

Screens have also an important application in the treatment of trade wastes. In fact, they are already used to a considerable extent in factories which handle fibrous material in a wet way. In England the writer has seen screens of the drum type or the cylindrical type operated so effectively as to produce a saving of material equal in value to the cost of operating them.

The author has made allusion to the extensive development of the art of screening in Germany. This may be illustrated by the following statistics collected by Mr. Sylvester Schattschneider, of the Department of Sanitary Engineering, Harvard University, as a part of a study of the sanitary conditions of certain German rivers.

Tables 17 and 18 show the number of sewage disposal plants of various kinds above the intakes of the water-works named in Column 1.

Mr. Whipple.

TABLE 17.—Sewage Treatment Works Above Water-Works Intakes of German Cities Supplied with Filtered Surface Water.

(1) Locations of intakes.	(2) Coarse screens.	(3) Fine screens.	(4) Plain sedimentation.	(5) Coarse screens and plain sedimentation.	(6) Kremer apparatus.	(7) Emscher tanks.	(8) Chemical precipitation.	(9) Chemical precipitation and sedimentation.	(10) Contact beds.	(11) Sprinkling filters.	(12) Contact beds and sprinkling filters.	(13) Broad irrigation.	(14) Intermittent sand filtration.	(15) Broad irrigation and intermittent sand filtration.	(16) Total number of cities and villages with sewage treatment works.	(17) Total population above intake, in thousands.	(18) Total population with sewage treatment, in thousands.	(19) Percentage of total population using sewage treatment.
Altona	4	1	18	3	3	6	4	2	4	7	1	12	2	0	67	22 680	6 344	28.0
Berlin (Mügel)	0	0	0	0	0	0	0	0	0	0	1	1	1	0	3	800	105	13.0
Berlin (Tegel.)	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1	266	12	4.5
Braunschweig	0	0	0	0	0	0	0	0	2	1	0	0	0	0	3	99	25	25.0
Bremen	2	1	7	4	1	1	0	1	5	3	1	4	2	1	33	4 133	1 937	47.0
Breslau	0	0	3	0	0	0	0	1	4	5	0	2	1	0	16	3 345	240	7.0
Brieg	0	0	3	0	0	0	0	1	1	5	0	2	1	0	13	2 310	208	9.0
Frankfurt	0	0	6	0	0	1	0	2	7	8	0	8	1	0	33	6 496	1 094	17.0
Glückstadt	4	1	18	3	3	6	4	2	4	7	1	12	3	0	68	23 023	6 350	28.0
Güstrow	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	16	0	0
Hamburg	3	1	18	3	3	6	4	2	4	7	1	12	2	0	66	21 545	5 412	25.0
Hannover	0	1	0	1	0	0	0	0	0	1	0	0	0	0	3	434	54	12.0
Königsberg	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	......	0	0
Lübeck	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	29	0	0
Magdeburg	2	1	10	2	1	3	0	2	5	7	0	3	0	0	36	15 890	2 247	14.0
Posen	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	2 635	25	1.0
Ratibor	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	840	0	0
Rostock	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	91	18	20.0
Schwerin	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
Stralsund	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
Stassfurt	0	0	0	1	0	0	0	0	1	2	0	1	0	0	5	239	113	47.0
Stettin	2	0	9	3	0	2	0	2	8	9	0	8	1	0	44	11 651	1 434	12.0
Stuttgart	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	459	4	0.9
Wandsbeck	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0
Worms	2	2	4	3	0	1	1	0	0	1	0	5	1	0	20	8 090	871	11.0
Zürich	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	98	......	......

Mr. Whipple.

TABLE 18.—SEWAGE FILTRATION STATISTICS OF GERMAN CITIES OF 10 000 OR MORE INHABITANTS ABOVE DIFFERENT INTAKES.

(1) Locations of intakes.	(2) Number of cities above intake.	(3) Cities without treatment.	(4) Coarse screens.	(5) Fine screens.	(6) Plain sedimentation.	(7) Coarse screens and plain sedimentation.	(8) Kremer apparatus.	(9) Emscher tanks.	(10) Kremer apparatus and Emscher tanks.	(11) Chemical precipitation.	(12) Chemical precipitation and sedimentation.	(13) Contact beds.	(14) Kremer apparatus and contact beds.	(15) Sprinkling filters.	(16) Contact beds and sprinkling filters.	(17) Broad irrigation.	(18) Intermittent sand filtration.	(19) Broad irrigation and intermittent sand filtration.	(20) Foreign cities on German catchment areas. (Information not available.)
Altona	118	41	3	1	9	1	2	2	0	3	2	3	0	5	1	10	2	0	28
Berlin (Mügel)	4	1	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	0	0
Berlin (Tegel)	3	1	0	0	1	0	0	0	0	0	0	0	0	0	0	1	0	0	0
Braunschweig	1	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
Bremen	24	9	1	1	3	3	0	0	0	0	1	0	1	1	0	3	0	1	0
Breslau	15	7	0	0	2	0	0	0	0	0	1	1	0	2	0	1	1	0	0
Brieg	15	8	0	0	2	0	0	0	0	0	1	0	0	2	0	1	1	0	0
Frankfurt	32	16	0	0	5	0	0	1	0	0	1	2	0	1	0	5	1	0	0
Glückstadt	115	43	3	1	9	1	2	2	0	3	2	3	0	5	1	10	2	0	28
Güstrow	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hamburg	111	40	2	1	9	1	2	2	0	3	2	3	0	5	1	10	2	0	28
Hannover	2	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Königsberg	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Lubeck	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Magdeburg	85	33	2	1	6	0	1	2	0	0	2	3	0	5	0	2	0	0	28
Posen	6	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	4
Ratibor	5	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	4
Rostock	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
Schwerin	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Stralsund	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Stassfurt	4	2	0	0	0	0	0	0	0	0	0	0	0	1	0	1	0	0	0
Stettin	44	17	2	0	7	0	0	2	0	0	1	2	0	2	0	5	1	0	5
Stuttgart	3	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Wandsbeck	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Worms	40	19	1	2	0	2	0	0	1	0	0	0	0	0	0	2	0	0	13
Zürich	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Mr. Whipple. In general, these have been classified according to the final process used in the treatment. The simplest form of treatment is that mentioned in Column 2, namely, coarse screens. The efficiencies of the treatments increase in order from left to right, the best effluents being obtained by broad irrigation and intermittent sand filtration. The figures do not indicate the number of screens in use, but only the number of plants where screening is the final process.

The figures include only cities which are in Germany. Some of the more important catchment areas extend into neighboring countries, so that the total number of treatment works above a given intake may slightly exceed the figures given.

Germany has followed the general principle of some treatment for all sewage, and for each the simplest treatment that will suitably protect the waterways. It will be noticed, also, that Germany does not depend on sewage treatment to protect water supplies, but on water filtration and the use of ground-water. The writer believes that, for many sections of the United States where the waterways are large, this is the best general principle to follow. The treatment selected for any given case must be adequate for the most adverse conditions, and it should not be forgotten that in many sections of the United States the streams reach a minimum flow far below that of German rivers, and also that the summer temperatures of the water in the United States are in general higher than in Germany and England. The German rivers differ from those of England in having swifter currents. Consequently, a simpler form of sewage treatment is permissible.

During the summer of 1914, Mr. Theodore R. Kendall, a recent graduate of the School of Engineering of Harvard University, has visited many of the sewage disposal plants of Europe, and the writer takes pleasure in presenting the following notes on visits which he made to places not mentioned by the author.

Eberswalde.—At Eberswalde two sets of screens precede the settling and biological treatment; the first set consists of round bars 2 cm. in diameter and with 3-cm. spacings. This arrests the coarser material and paper, and is cleaned by hand when necessary. Immediately beyond is a Riensch-Wurl screen with 3-mm. openings. All screenings are elevated to the surface and discharged into metal cans holding about ½ cu. yd. each. The screenings sell for about 2.5 cents per can. There was no odor about this plant when the temperature was 35° cent. (95° Fahr.). Stadtbaurat Arndt mentioned that before screens were installed the sedimentation basins had to be cleaned every 3 weeks; now they can be used from 4 to 5 months. Population, 37 000.

Düsseldorf.—The sewage of Düsseldorf is approximately 1 cu. m. per sec., giving a velocity of 0.78 m. per sec. in the main outfall. The flow passes a *grobrechen* or coarse screen with six wings, one of which

Mr. Whipple.

is in the sewage; when this becomes slightly clogged, an attendant turns it and removes the screenings. These wings are about 3 m. long. The fine screens are of 2-mm. piano wire with 2-mm. spacing, and are on large steel frames. There are six of these screens, three of which are sufficient for ordinary flows. The fine screens are cleaned by rotating arms which rake off all screenings, and are in turn cleaned by a scraper which removes the screenings to a continuous conveyor. The average quantity of screenings is 13 cu. m. per day (10 cu. m. between 8 A. M. and 9 P. M., and 3 cu. m. from 9 P. M. to 8 A. M.). The plant has been in operation 10 years, and costs about 40 000 Marks ($10 000) per year for operation and repairs. The screened sewage contains only ½% of sediment after standing 2 hours. To operate all screens requires only 6 h.p., which is furnished by a steam engine capable of producing 15 h.p. The screenings are piled in large concrete bins having under-drains, and are covered with *torf* and chalk to keep the odor at a minimum. The screenings sell in October for 1.50 Marks per 1 000 kg. (36 cents per ton). Population, 400 000. Sewered area, 25 000 acres.

Cologne.—The sewage is screened by 0.5 and 2.5-mm. screens, and these are cleaned by wire brushes which carry the screenings to an endless conveyor and thence to a car which removes them to concrete pits for storing, as at Düsseldorf. The screenings sell at 0.50 Mark (12 cents) for a 2-cu. m. wagon load, and 0.30 Mark (7 cents) for a 1-cu. m. load. The odor here was nil at the screens, but very bad at the compost piles. The screens remove about 15 cu. m. per day from 75 000 cu. m. of sewage. Population, 440 000. Future works are to be built with Riensch-Wurl screens because of the greater accessibility of the whole screen and cleaning devices for repairs.

Wimbledon, England.—The screens consist of fixed bars, with engaging teeth to clean them, the latter being operated by motors. Similar screens are in use at Chester, Birmingham, Salford, Wakefield, and Huddersfield, at present, and differ widely from the Carshalton screen shown in this paper, inasmuch as the screen is fixed and the cleaning device movable, whereas the Carshalton screen itself moves and is cleaned at the top.

Rochdale, England.—The same general type of screen is in use in Rochdale as at Wimbledon, but the method of cleaning is slightly different and worthy of mention.

"The three screening and raking apparatus (Law's patent) are fixed in chambers 8 ft. wide and 6 ft. 9½ in. deep at the outlet end * * *. Each inclined screen is fitted with taper section steel bars 7 ft. 6 in. long, spaced ½ in. apart, the maximum sewage level being about 5 ft. The raking gear above the coping consists in each apparatus of a built-up steel framework * * * carrying a travelling-cleaning rake fitted with * * * malleable prongs. The rake is suspended by

Mr. Whipple. two vertical 'T'-section arms from a cross-bar on the chains, and is counterbalanced * * *. The ends of the rake are fitted with runner pulleys, working between cast-iron guides in front of the inclined screen. In working, the rake descends through the guides farthest from the screen, and ascends through the guides nearest to the screen, the prongs engaging in the spaces between the screen bars and removing the refuse therefrom. A vertical steel plate is fitted at the top of the screen to prevent the refuse falling from the rake, and, on the rake arriving at the highest point of travel, the refuse is automatically discharged by a cleaner, consisting of a set of vertical prongs in a cross-bearer, actuated from the detachable chains which carry the rake. At the junction of the 'up' and 'down' guides a hinged flap is fitted, to ensure the rake descending the guides farthest from the inclined screen."*

North Bierley, near Bradford, England.—Wool fiber and picric acid are the chief trade wastes in this locality, so a screen was designed to be as completely accessible as possible and still effectively remove the fibers. The screen consists of a perforated sheet of copper, about 7 ft. square and bent to form a half cylinder, along which the sewage flows. Revolving wire brushes pick up the lint and other solids and are in turn cleaned by a straight bar which passes over them when they reach the upper edge of the screen. This type of screen was adapted from the screens used in the mills near-by.

Rothwell Urban District Works, near Wakefield.—The use of a rapidly revolving disk screen at this plant, which receives a daily dry-weather flow of 54 000 gal. from 4 000 people, is chiefly to break up the solid matter before sending it to the trickling filter. This is probably the only plant of this type in existence. The sewage first passes a bar screen with ½-in. openings, and then is pumped up to the disk screen, which revolves rapidly in a horizontal plane, allowing the liquid and most solids which are well broken up to pass through the ¼-in. holes and rejecting practically only mineral and other heavy solids which are not easily broken up. These are caught in a tray at one side and removed to be buried, as the volume is very small. It may be well to add that the trickling filter bed is chiefly composed of broken brick with a 2-in. top covering of 1-in. gravel.

Colehall, near Birmingham.—This is a rather good example of a trickling filter installation working well without screens. This plant was constructed some years ago, but within a year has come under the control of the Birmingham Corporation. The sewage is practically identical with that of Birmingham. The population served is about 60 000, and the flow is 2 400 000 gal. daily. The flow is directed through three settling basins in series, the heaviest material, chiefly mineral.

* Institute of Municipal and County Engrs., June 14th, 1913, by the courtesy of Mr. S. S. Platt, Borough Surveyor.

settling out in the first and the finer in the latter basins. The basins are cleaned by a clam-shell bucket operated from a track at the side. The first basin is cleaned every 2 weeks, the second every 4 weeks and the third every 8 weeks. If a heavy storm occurs when the first basin needs cleaning, the material is simply flushed over into the next basin. At no time has there been any trouble with detritus being washed past the third basin. The results of the operation are: Suspended matter, raw sewage, 500 to 800 parts per million; influent to sprinklers, 50 to 120 parts per million; oxygen absorbed, raw sewage, 200 to 300 parts per million; influent to sprinklers, 50 to 90 parts per million. Mr. Whipple.

The author does well to emphasize the need of greater system in keeping the results of operation of sewage screens and of weighting the results of samples on the basis of the corresponding volumes of sewage flow when the samples were collected. He might have added, also, the need of more frequent sampling.

In spite of the greatest care in devising and maintaining systems of sample collection and analysis, the results will be understood by only a few. There is need of a simpler and more tangible record of suspended matter than that which is used.

An excellent method of keeping such a record is that of filtering samples of water through cotton disks. A given volume of water, say 1 gal., is filtered through a disk of absorbent cotton, about 1 in. in diameter, specially prepared for the purpose. This is most conveniently done with a simple apparatus known as the "Wizard Sediment Tester" which is made by the Creamery Package Manufacturing Company, of Albany. The cotton disk, which is the filtering medium, is held between two supports of wire cloth in a cap attached to an ordinary glass milk bottle. The water to be filtered is placed in the bottle and allowed to flow out through the cotton disk. Filtration is hastened by increasing the pressure of the air within the bottle by the use of a simple air compressor operated by a hand bulb. For samples of sewage, a much smaller quantity than 1 gal. is required. The cotton disk sediment record has the following advantages:

1. It can be made very easily and quickly;
2. It can be made by the laborer and the office boy as well as by the engineer or chemist;
3. It is inexpensive;
4. The records are picturesque, and easily understood by every one;
5. The records are permanent;
6. The records may be conveniently mounted for preservation;
7. The records may be photographed;
8. A relatively large volume of water is tested; and
9. It is a valuable supplement to the regular water analysis.

Mr. Jennings.

C. A. JENNINGS,* ASSOC. AM. SOC. C. E. (by letter).—Mr. Allen has compiled a mass of useful and interesting information and data on the subject of fine-screening of sewage. The writer has had much difficulty, in searching English publications, in finding very many reliable data on this subject. This paper will serve a very useful purpose. There is no doubt that in the United States fine screens have not received the attention they deserve. Whereas numerous experiments have been made on the various types of settling tanks, none has been made on the use of fine screens, so far as the writer is aware.

The Jennings screen was mentioned by Mr. Allen as a recently patented type of band screen. This screen is handling successfully in Chicago a typical stock-yards sewage, which is rich in corn, straw, and hay, and cattle, hog, and sheep manure, and there is also present some human sewage. It comes from an area of some 30 acres, on which are live-stock pens, streets, buildings, etc. Formerly, the screen was cleansed by air under a pressure of about 45 lb., delivered through $\frac{3}{64}$-in. perforations in a $1\frac{1}{2}$-in. pipe. These holes became clogged with rust from the air pipe, and the high pressure of the air wore out the screens rapidly. This has been entirely remedied by supplying about 500 cu. ft. of air per min., under a pressure of $1\frac{1}{2}$ lb. per sq. in., through a slotted 3-in. pipe. The air is supplied by a positive blower driven by the same motor that drives the screen. This has overcome the objections to the high-pressure air, and, at the same time, has increased the efficiency of the screen. The data in Table 19 are from a forthcoming report of the Sanitary District of Chicago, dealing with the sewage problem at the Chicago Stock Yards. The writer is indebted to George M. Wisner, M. Am. Soc. C. E., Chief Engineer, and Langdon Pearse, M. Am. Soc. C. E., Division Engineer, for their use, the data having been collected under their direction.

Mr. Allen mentions the high moisture content of the screenings. This was due, at that time, and in the data in Table 19, to the method of constructing the screen frames of this first installation. Due to particles of corn, coal, gravel, etc., rolling down the screen, a 2 by 2 by $\frac{1}{8}$-in. angle-iron, with an outstanding leg pointing up on the lower side, was substituted for the $\frac{3}{4}$ by $\frac{1}{8}$-in. plate. This angle-iron retained the larger particles which formerly rolled down the screen, but it also retained considerable water until the frame reached the top of the incline where the blast of air blew off the screenings. This trouble has now been overcome by perforating the angle-iron with $\frac{1}{2}$-in. holes, so that the water drains out before the screen reaches the top of the incline. A number of tests made recently showed the average moisture content of the screenings to be 73.5%, as against a former average of 79 per cent. These wet screenings are easily handled with

* Chicago, Ill.

a shovel or pitchfork. They are dark green in color, and, while fresh, the odor is inoffensive. Table 20 gives typical analyses of the screenings. Mr. Jennings.

TABLE 19.—CRUDE SEWAGE.

Test No.	Rate of flow of sewage.	Gallons per square foot of screen exposed.	MOIST SCREENINGS.		DRY SCREENINGS.
			Pounds, per million gallons.	Percentage of moisture.	Pounds, per million gallons.
1..........	1 120 000	2.11	6 170	83.0	1 050
2..........	1 138 000	2.14	4 820	80.4	945
3..........	1 440 000	2.71	7 860	82.0	1 420
4..........	1 305 000	2.45	7 390	86.6	990
5..........	1 208 000	2.37	7 200	82.5	1 260
Average...	1 241 000	2.36	6 690	82.9	1 130

NOTE.—With screenings weighing 55 lb. per cu. ft., the volume of screenings varied from 3.2 to 5.3 cu. yd., and averaged 4.5 cu. yd. per million gallons of sewage screened.

TABLE 20.

Percentage of moisture.	CALCULATED TO DRY WEIGHT—PERCENTAGE.			
	Nitrogen.	Volatile matter.	Fixed matter.	Ether, soluble.
83.0	2.24	93.0	7.0	4.1
80.4	1.60	94.0	6.0	3.0

NOTE.—These data are from a forthcoming report of the Sanitary District of Chicago on the Stock Yards sewage problem.

On a recent installation, the percentage of moisture in the screenings has been reduced by reversing the angle-iron referred to, so that it carries no water with it, the water draining through the screen immediately.

The writer is of the opinion that percentage-removal data mean little. Probably the technique of this determination differs in every laboratory. When one considers the immense quantity of sewage screened, the very small portion of this volume going to make up the composite sample, the small quantity of the composite sample used for the determination, and finally the size of the particles in suspension in the raw sewage, it is easy to see many chances for error in making this determination. The writer has seen a chemist use a 10-cu. cm. pipette to collect a sample of raw sewage to determine the suspended matter, in spite of the fact that very few of the particles in suspension could be sucked up through its tip. Not less than 40 or 50 cu. cm. should be used in making this test, and the sample

Mr. Jennings.

should be collected in a graduate, in the top of which a funnel is placed. The bottle of sewage is shaken in an inverted position above the funnel and graduate, and the stopper or one's hand quickly withdrawn and sufficient sewage allowed to enter the graduate through the funnel. This method has been tested by the writer and found to give comparable results.

Sewage from about thirty-five packing houses enters Bubbly Creek, or the Stock Yards Slip, a branch of the Drainage Canal. This sewage is quite strong. It contains considerable fat and particles of skin and hair which are not intercepted in the catch-basins of the various packing houses. The Jennings screen is now handling a similar sewage in another city. At Chicago, this packing-house sewage receives no treatment other than the passage through catch-basins. It is with this problem that the Sanitary District of Chicago has experimented for about two years, and a report is now being prepared on the results of this work. It seems to the writer that here is an excellent field for fine-screening. The water from the Drainage Canal is not used for domestic consumption, and, furthermore, the particles that can be removed by fine screens are those which would settle to the bottom of the canal to a great extent and form deposits. In this way much of the small quantity of dissolved oxygen in the canal would be used up. It is desirable to remove this matter in suspension, and a cheap, effective, and quick method of doing so is by fine-screening.

Other fields for fine screens have appeared in manufacturing plants where the problem is either to recover something in suspension which has value, or to prevent something in suspension, which has no value, from being discharged into a stream or other body of water. The more the writer has gone into this subject, the more he is convinced that fine-screening of sewage has been overlooked to a large extent, and that there are many places where its application is as good as, or, in some instances, even preferable to, tank treatment.

Messrs. Franze and Schaefer.

MESSRS. G. FRANZE* AND HERMANN SCHAEFER† (by letter).—The author's painstaking work forms an important contribution in settling the question of the importance of screening installations in the field of sewage clarification. While acknowledging the correctness of Mr. Allen's deductions and conclusions, the writers beg to present some additional observations founded on their experience.

In considering a plant about to be installed, one should provide for the most complete treatment that may be demanded, if he wishes to satisfy the variety of requirements that may arise eventually for the cleanliness of the stream. A plant, therefore, should be planned and provided from the beginning with everything which later may be

* Stadtbaurat, Frankfort, Germany.

† Stadtbaumeister, Frankfort, Germany.

necessary for the kind of treatment which has proved satisfactory in all cases. Its development in accordance with the more complete demands of the future can then be temporarily postponed, provided the conditions are such as to make this advisable. In such cases, the first installation of the plant should provide for mechanical cleansing by screens (bars, sieves, grids, etc.). Messrs. Franze and Schaefer.

This procedure is to be preferred under the following circumstances:

(1).—As an independent and separate arrangement for the removal of suspended matter greater than a given size where the stream to which the effluent passes is large, thereby providing ample dilution, and where there are no communities below which require the use of the water; and also where there are æsthetic reasons for preventing the fouling of the shores.

Such independent arrangements, however, have no influence on the removal of dissolved impurities. These are disposed of by the biological purifying agencies contained in the stream itself, as long as the volume of sewage remains within certain limits relative to the self-cleansing ability of the stream.

(2).—As a preliminary clarification plant in connection with other processes for the further removal of the finest suspended matter. In this case screens form also an independent and necessary feature:

(*a*) During periods of flood in the stream;
(*b*) Without affecting unfavorably further clarification;
(*c*) Without preventing the subsequent treatment of the sludge by machinery.

(3).—As a protection to pumps in systems of low-level sewers.

The conditions affecting plants mentioned under (1), (2), and (3), depend further on the kind and composition of the domestic and factory wastes. The best and most efficient results are with fresh sewage. The most unfavorable are with stale sewage, where the impurities reach the plant in a finely broken up and decomposed condition. The most important advantages in mechanical treatment are the following:

(1).—The product contains only about 75% of moisture and is, therefore, in its firm condition, capable of transportation. The quantity removed, if carried to tanks or other arrangements, would increase the quantity of sludge by from 10 to 15 per cent.

(2).—The fertilizing value of the material is greater, thus paying for its carriage to a greater distance.

(3).—The removal of coarse and clogging material renders possible an unobjectionable and certain removal of sludge from the tanks which, in some cases, must be provided with complicated pipe systems for its removal either by suction or pressure.

Messrs. Franze and Schaefer.

(4).—This also holds good in the further mechanical treatment of the sludge.

(5).—The simple and automatic machinery requires only a small sum for service and power.

With regard to the agricultural use of material removed by mechanical devices, it was formerly given to neighboring farmers. After experiments made at Frankfort, Germany, however, it is now possible to dry the product by the chimney gases of the refuse incinerator so completely, in a drying drum, that the heat developed in burning can be utilized for electric power, or the material can be used on farm lands as poudrette.

The choice between the successful mechanical plants now known—which may be divided into two groups, fixed and movable—should be made in each case only after a thorough study of local conditions as well as of the character of the sewage and the stream into which it is discharged.

Mr. Fuller.

GEORGE W. FULLER,* M. AM. SOC. C. E. (by letter).—Mr. Allen has written a good paper on a subject about which practical knowledge in the United States falls far behind that which is available in Europe. Undoubtedly, his conclusion is correct, that fine screens for sewage clarification are entitled to more consideration than they have generally received hitherto in America.

As a general proposition, it may be safely stated that fine screens have their legitimate field of usefulness where the clarification effected by them can be accomplished at less cost than by other available methods, when due consideration is given to capital charges and depreciation, as well as to ordinary operating expenses and current maintenance. In some places where the total cost of screens might be somewhat greater than that of settling tanks, the convenience may justify their use.

Fine screens are most serviceable in cases where clarification alone or in combination with sterilization is sufficient treatment to give to the sewage. Where settling tanks are used, fine screens do not seem to be of sufficient aid to justify their cost. Perhaps exceptions may be found in instances where some by-products of value are to be found.

In connection with filters of various types, fine screens do not seem to have established themselves as of much practical aid, if exception be made of cases like Baltimore, where gas ebullition lifts much sludge at times from the single-story tanks on to the filters. The experience of Brockton is of much interest, and it would be helpful to know how expenditures for fine screens there would compare with what could be accomplished by modern settling tanks, in lessening the load on the sand filters.

* New York City.

Last spring, the writer designed a sewage treatment plant at Daytona, Fla., in which Riensch-Wurl screens (with a width of opening of $\frac{1}{12}$ in.) and sterilization afford the only treatment before the dispersion of the sewage in the Halifax River not far distant from shell-fish layings. It was concluded that these fine screens would give sufficient clarification and that the cost would be materially less than for settling tanks for a city where quicksand underlies practically the whole area, with its upper surface from 6 to 12 ft. below the surface of the ground and extending to depths of some 30 to 40 ft. Mr. Fuller.

Mr. Allen's paper has much value for reference, on account of the European data which it presents. The writer believes, however, that actual experience with fine screens in the United States is likely to show a smaller removal of suspended matter than would be inferred from statements in the paper. Thus, at Reading, Pa., there is quoted a removal of dry matter equal to 90 parts per million, whereas, for several years, the writer's studies indicated that, on a basis of daily averages, this removal rarely if ever exceeded 30 parts per million. Without doubt, the comparatively dilute American sewages cannot be clarified with the same relative efficiency by fine screens as the concentrated sewages of Europe. As the sewage becomes comminuted, through age and mechanical attrition, incident to flowing through long outfall sewers, the efficiency of fine screens will become less.

Generally speaking, fine screens have not been built strongly enough, either in the United States or in the earlier installations in Europe, to do their work satisfactorily without more or less overhauling and reinforcing. Without such heavier construction, the periods of interruption and the cost of attendance become relatively great. There should be due allowance for this factor in making a preliminary estimate of cost.

Notwithstanding the foregoing comments as to lessened efficiency, as compared with the author's statements, and the likelihood of greater cost, the writer believes firmly that in the future fine screens will play a prominent part in solving many problems of sewage treatment in America.

J. X. Cohen,* Jun. Am. Soc. C. E. (by letter).—The author deserves credit for the presentation of data (undoubtedly collected with much effort) which enrich the meager American literature on sewage screening. He has brought to the attention of American engineers a process of sewage treatment which merits more attention than it has received as yet in the United States. Mr. Cohen.

Most sewage screens in the United States are of the crude bar type, and although they serve their intended purpose more or less effectively, there are without doubt a number of plants where improved

* New York City.

Mr. Cohen. screens, if planned with the painstaking care shown in most of the designs presented in the paper, would give better results.

The success of the mechanical screens discussed by the author, however, will become possible in America only when more of our municipalities come to hold enlightened views regarding the necessity for skillful operation and scientific supervision in sewage treatment works. It will be of little avail to install an expensive screen of high efficiency, and then allow its control to be vested in an operator of the type so usual in numerous American sewage disposal works. Whereas the methods and structures now in use are capable of more abuse in practice, without great deterioration in the quality of the effluent, than are mechanical screens, and in consideration of the present attitude of many public officials, that sewage disposal works are necessary evils, frequently forced on them by Court orders, which must be tolerated, but to which as little attention as possible should be given, sanitary engineers who realize the conditions fully, will hesitate to recommend the installation of automatic screening machinery.

On the other hand, it may be that the installation of such screens in progressive communities, where problems of municipal management of public works are being solved, not from the viewpoint of political exigency, but from that of scientific reliability, will present concrete object lessons in economical municipal management that in time will be learned. Without this recognition of the importance and the vital necessity for reliable sewage works operation, however, much of the effort and expense involved in the design and installation of high-efficiency, sewage-screening machinery will be lost.

The writer is of the opinion that, in the design of sewage screens, the convenience of the attendant has not always been kept in mind. This opinion is based on an experience of about 8 years, and refers in particular to small plants. The result of this lack of prevision is that screens are either improperly cared for, neglected, or are completely removed from the sewage flow.

As it is a well-known fact that many small sewage disposal works are poorly conducted and therefore produce effluents which are not wholly satisfactory, it should be the duty of the designing engineer to arrange the various parts of the plant so that in their operation the attendant, at all times, will be as free as possible from arduous labor in awkward positions. As an instance of the apparent failure to keep in mind during design the practical operation of a screen, there may be mentioned a small disposal works near New York City where the operator has found the task of cleaning the screen so difficult, because of its position, that it has been neglected for a long time, with the natural result that it clogged and the sewage on its way to the pumps overflowed the screen. Thus far, it is not

known that this has caused any trouble at the pumps, but, of course, deposits have accumulated above the screen, with the accompanying nuisance. As this particular screen-chamber is covered, it will be some time, perhaps, before the odors of decomposition become sufficiently obnoxious to force the removal of the deposits. Mr. Cohen.

In another small plant, the basket-bar screen is too heavy and cumbersome for the lone operator to handle conveniently, and the result has been the withdrawal of the screen from the well. The sewage now flows unscreened and uninterrupted to the pumps, thereby endangering their safe and continuous operation. Such practices do not enhance the respect of the operator for the engineer, nor redound to the credit of the latter.

There is a feature of fine sewage screening that will be of service to its development in America, which deserves considerable attention. Centrifugal pumps are now used extensively in pumping sewage, and in most recent designs other types have not been given equal consideration. Fine screens will eliminate the grosser solids found in coarse screened sewage, and the manufacturers of centrifugal pumps will be enabled to design impellers and pump casings which will allow the pumps to attain higher efficiencies than are now feasible. For large flows, the saving in the cost of pumping thus obtained will be a factor which should have considerable and favorable influence on the decision concerning the use of mechanical screens.

Mechanically operated fine screens in large plants probably receive the skilled attention which they require, but in smaller plants, where the quality of the effluent may be equally important, if not more so, such screens may not always receive as good attention. Hence, it behooves engineers to give sufficient study to the design of sewage screens, whether of the simple stationary bar type or the more complex mechanical type, in order that, under practical operating conditions, they may receive more than sporadic attention. Such attention, or rather lack of attention, forces the other parts of the disposal works to exercise functions for which they are not designed, with the consequent lowering of their efficiency. By presenting to the Society data which indicate how other engineers are ingeniously working out the problems of sewage screening, the author has made a noteworthy contribution which should stimulate other members to equal endeavors.

F. T. Robson,* Assoc. M. Am. Soc. C. E. (by letter).—While engaged in the design of a sewage disposal system in Southern California, the writer heard of the great difficulty that The Anaheim Sugar Company was having in disposing of its waste water which carried a large percentage of sugar-beet waste. The factory was in a thickly settled, highly developed farming district containing many small cities, Mr. Robson.

*San Francisco, Cal.

Mr. Robson. and at such a distance from the ocean as to make it an economic impossibility to discharge the waste water there. It might have been delivered directly to the citrus orchards immediately adjoining, or into a ditch for irrigating lands at a greater distance. Most of the farmers refused to use the water, as the factory operates for the entire 24 hours of the day during a "campaign", and night irrigation, as well as painstaking attention to cover or plow under the large quantity of suspended matter, would thus be necessary.

The factory had been completed and put in operation in 1911, and the campaign of 1913 was fast approaching when the writer was asked to report on the problem. In 1911 the wastes were discharged in an artificial 55-acre lagoon near the plant, and readily decomposed under anærobic action, producing much hydrogen sulphide. Many serious complaints from near-by residents resulted. In 1912 the wastes were discharged into an irrigation ditch, but the large quantity of organic suspended matter caused difficulty similar to that in the previous year. Complaint was made to the State Board of Health, so, in 1913, the company was compelled to abate the nuisance or close the factory. It was essential that something be done at once, for the time was short.

In local parlance the word "campaign", used in connection with a sugar factory, means the period from the time when the first beets are received, or the factory is started, until the last beet has been cut up and the factory closed. During this time the factory runs 24 hours per day and a shutdown is a serious matter. All energies during the "closed" period are devoted to putting everything in such shape as to minimize the chance of a breakdown.

After careful study the only available methods of solving the problem seemed to be:

(1) Fine screening;
(2) Sedimentation.

The dearth of information regarding recent installations of fine-screening devices, their cost, operating difficulties, etc., was astonishing. For this reason Mr. Allen's paper is especially valuable, and should be of wide interest, for fine screening is available as a method of sewage disposal in a large number of cases. He has collected in one paper far more information, as to design and operation, than was found by the writer in a number of authorities consulted.

On investigating this local matter, many aspects differing from the usual sewage disposal problem were found, and Charles Gilman Hyde, M. Am. Soc. C. E., Professor of Sanitary Engineering, University of California, was asked to assist. His report was very detailed and comprehensive, and of great value, because it was the first time in California and probably in America that the disposal of beet

sugar factory wastes had been studied scientifically. Tables 21, 22, etc., are taken from Mr. Hyde's report to the writer. Mr. Robson.

TABLE 21.—CHEMICAL ANALYSIS OF ROUGHLY SCREENED BEET SUGAR WASTE.

Determinations.		Parts per Million.
Solid matters	Total	2 588.0
	Mineral	1 521.6
	Organic and volatile	1 066.4
Suspended matters	Total	1 891.2
	Mineral	1 208.0
	Organic and volatile	683.2
Dissolved matters	Total	696.8
	Mineral	313.6
	Organic and volatile	383.2
Nitrogen (N) as	Organic nitrogen	7.14
	Free ammonia	5.60
	Albuminoid ammonia	1.80
	Nitrites	0.19
	Nitrates	0.22
	Total	13.15
Oxygen consumed (O)	"Absolute"	1 985.
	5 min. at 100° cent.	496.
	20 min. at 80° cent.	198.50
Sulphates (SO_3)	Total (dissolved)	50.60
Chlorine (Cl)	Total (dissolved)	40.00
Phosphoric acid (P_2O_5)	Total (dissolved)	Trace.
Potash (K_2O)	Total (dissolved)	10.05
Soda (Na_2O)	Total (dissolved)	43.50

As far as known, this installation is the first in which sugar factory waste has been treated by fine screening; it is also the first installation of a screening device of any kind in California. A brief description may be of interest.

Investigations preliminary to design showed that the volume of waste requiring treatment was about 2 200 000 gal. daily during the campaign (approximately from August 1st to November 1st).

General Character of Waste.—The waste consisted of discolored water heavily loaded with suspended matter most of which was heavier than water—some lighter; it was largely sand, silt, clay, beet tops, roots, cuttings, etc., the impurities from beets or unpurified sugar, and the sewage from 200 employees.

Chemical Analysis of Waste.—The result of a chemical analysis of roughly screened waste, made at the University of California, is given in Table 21; and from this it follows that the suspended solids were equivalent to about 15 800 lb. per million gallons.

Analysis of Water.—The mineral composition of the water carrying the waste is shown by Table 22.

Mr. Robson.

TABLE 22.—ANALYSIS OF WATER CARRYING WASTE.

Determinations.		Parts per Million.
Total solid matters	Total	397.0
	Mineral	357.0
	Organic and volatile	40.0
Suspended matters	Total	0.0
Dissolved matters	Total	397.0
	Mineral	357.0
	Organic and volatile	40.0
Sulphates (SO_3)	Total (dissolved)	22.2
Chlorine (Cl)	Total (dissolved)	34.5
Alkalinity due to sodium carbonate		6.4
Incrusting solids:		
	Calcium and magnesium carbonate, etc., chiefly small; Calcium sulphate (Gypsum)	244.0
	Silica (SiO_2)	10.0

Volume of Suspended Matter.—(1) By Imhoff Graduates.—

Time of subsidence.	Average volume.
1 min	6.3 cu. cm.
2 "	6.9 "
3 "	7.1 "
4 "	7.2 "
5 "	7.3 "
10 "	7.4 "
15 "	7.6 "

This is equivalent to 37.8 cu. yd. per million gallons, or, for the daily flow of 2 200 000 gal., a total of 83.2 cu. yd. of wet sludge having a moisture content of about 95%, giving a total dry weight of 3 170 lb. per million gallons, or 34.87 tons per day after 15 min. sedimentation. Computing the total suspended solids as 15 800 lb. per million gallons, the quantity capable of settling in 15 min. would be about 20 per cent.

Volume of Suspended Matter.—(2) By Screening.—Brass sand-analysis sieves, of 40 and 100 mesh, in tandem, gave the following results:

On the 40-mesh screen, from 17 240 cu. cm. of waste, 71.3 grammes wet, or 3.7 grammes dry, were caught; or, on the basis of 15 800 lb. per million gallons, the percentage was 11.3%, or an estimated quantity of 46.6 cu. yd. of wet material per day.

On the 100-mesh screen, 11.4 grammes wet, or 0.9 grammes dry, were retained; or, on the basis previously given, about 14.0% was retained with both screens.

Although these percentages appear to be low, the material removed represents that portion of the waste matter that caused the trouble by decomposition. Mr. Robson.

Location of Plant.—The factory is on an old river bed or channel of sand, which is very fine in the top layers, but grows coarser with depth. Tests showed the perfect feasibility of intermittent filtration, after the top layers of sand were removed. It was thus determined that some process of preliminary treatment followed by sand filtration must be used. The following figures give an idea of the estimated cost of the complete plant of the two types best adapted to the work:

	Cost of construction.	Annual operating expenses.
Fine screening and intermittent sand filtration	$32 500	$6 600
Sedimentation in Imhoff tanks and intermittent sand filtration	84 500	8 900

These annual operating costs include interest on the investment at 6%, depreciation, maintenance, labor, and power costs.

The fine-screening method, with a battery of two screens, having been recommended, plans were prepared. The company, however, expressed some doubts as to the success of the plant, and decided to invest in only one screen for the present, though space was provided in the screen-house for two.

A Weand rotary screen of the Atlanta type, furnished by the Merritt Hydraulics Company, was put in. It is cylindrical, 10 ft. long, 8 ft. in diameter at the inlet and 6 ft. at the outlet, and is covered with a 40-mesh screen. It is chain-driven from a 5-h.p. motor at a speed of 10 rev. per min. The lower half of the inlet end is closed, but there is an opening for the supply pipe. The water entering the screen filters through the bottom portion, and the suspended matter is carried by worm plates to the outlet and deposited by a chute in an iron push-car with a perforated bottom. The screen is kept clean by water jets or nozzles.

The screenings are dumped on an open compost heap, where they gradually dry, with no disagreeable odor. When dry, the compost is used by the farmers as a fertilizer.

The screened water passes from the screen to a sump and thence by gravity to thirty-six filter beds, each 100 by 200 ft., or a gross area of 26 acres. These beds are covered once in 24 hours with about 4 in. of the screened water. They perform their function satisfactorily, and no odor is apparent. The plant is now going through the second campaign, and the results have been very successful.

The screen itself, however, has given a great deal of trouble. It was doubtless the first one manufactured by the present makers and,

Mr. Robson. at the time of purchase, they had not fully completed their arrangements with Mr. Weand's heirs. The mechanical details of the screen were very poor, and for this reason the operating delays were numerous. In fact, the screen was almost completely rebuilt during the season. Notwithstanding annoyances and delays of this nature, the company has been well satisfied, and the total capital expense for the complete plant, to date, has been about $30 000.

The cheapest alternate method of disposal would doubtless have cost three times as much, could not have been finished in time for the 1912 season, and would have been more expensive to operate.

The wet sludge or screened material has been found to contain about 92% of moisture and to weigh about 45 lb. per cu. ft., as it comes from the screen, without compacting. The quantity of sludge varies largely with local conditions in the factory, but the volume per day or week checks closely with that estimated.

Mr. Potter. ALEXANDER POTTER,* ASSOC. M. AM. SOC. C. E.—The speaker would like to add a tribute to the thoroughness with which the author has presented this subject. Although experience with fine sewage screens in the United States is limited, this is not true of Europe where fine screening is in more general use. This is especially true in Germany, where, since 1903, the Government has sanctioned the discharge, into large bodies of water, of sewage from which all suspended matter more than 3 mm. in diameter has been removed.

The principal contributing causes for the relatively small use of fine screens in America, as compared with their use in Europe, appear to the speaker to be the following:

First.—The failure of competent authority to establish reasonable regulations as to what would constitute adequate treatment under all conditions. There has been too great a tendency to permit the gross pollution of waters by sewage for much longer periods than are desirable, and these periods have been lengthened unconscionably by the insistence of too many of the State Boards of Health on standards of purification which are too high. Consequently, simple processes for removing the grosser pollution have not been received with favor.

Second.—European experience with the cost of operation and maintenance of fine screens indicates that these items will be exceeded by fully 100% in American practice.

Third.—As most purification works which are projected demand more than one step in the process, fine screens have not seemed capable, for a given cost, of producing results as satisfactory as those obtained with settling tanks of the latest design.

The percentage of suspended matter in the sewage removed by fine screening, as compared with settling, is not a true measure of

* New York City.

the relative efficiencies. For the same percentage of suspended matter removed, the screened liquid is far inferior to the settled effluent. The truth of this assertion is amply verified by an examination of Table 4. According to this table the suspended solids before screening amount to 2 112.8, and after screening they amount to 785.2—a reduction of 62.9%; whereas, on the other hand, the residue of the filtered liquid after evaporation has been increased from 1 237.2 to 1 260.4. The speaker is of the opinion that the author's statement, that the organic matter in solution has been increased only 1.9%, is in error. From both quantities should be deducted the total stable solids in solution, these solids being the same in each case. Deducting these, the relation between the organic impurities in solution before and after screening will show a much greater percentage of increase than that stated by the author. Mr. Potter.

It would seem that in the fine-screening process a portion of the solid matter is converted into colloidal matter in its passage through the screen. As this colloidal matter will pass through filter paper, the screens are credited with a greater percentage of removal than they are entitled to. The transformation of a certain portion of the suspended matter in the sewage into colloids during the process of screening increases the organic impurities in the liquid to a very material extent.

To bear out this statement, the speaker quotes from Mr. John D. Watson's report of 1912 in discussing results obtained at Ashold and Sutton. At this plant, the sewage passes through grit-chambers and receives a preliminary treatment by passing through two Carshalton screens. As to the efficiency of these screens, Mr. Watson says:

"Some time ago one of the Carshalton screens required renewal. This, however, I decided not to do owing to the discovery that the sewage contained more organic impurities *after* it was screened than it did before, thus adding unnecessarily to the work of the bacteria beds. Mr. O'Shaughnessy's report stated that 'Fæcal matter forms a colloidal solution when agitated with water; and the increase in the oxygen-absorbed figure, together with the increase in the colloidal matter present in the sewage liquor after passing the screen, showed conclusively the objectionable action of the screen'. The object of eliminating solids is to reduce the strength of the liquid, but to brush particles of fæces over the perforations of the screen and afterwards to force sewage through those perforations must tend to defeat the object in view. The truth of this forced itself upon me in viewing what is perhaps the most complete and elaborate screen I have ever seen, viz., that by Riensch at Dresden."

George M. Wisner, M. Am. Soc. C. E., discovered the same phenomena at the Thirty-ninth Street testing station in Chicago, as mentioned in his report of 1911 on "Sewage Disposal for the Sanitary District of Chicago".

Mr. Potter. It is the speaker's opinion, however, that where dependence for sewage disposal must be placed chiefly on dilution, there is a wide field for fine screening, for in that case the object to be attained is principally the removal of the rather large particles of suspended matter.

Where a high degree of purification is sought, the use of fine screens is of doubtful value. A modern settling tank will give better results and at less cost for a given degree of purification. A settled liquid also is superior to a screened liquid for subsequent biological treatment in filters. Furthermore, a settling tank can handle septic sewage or a sewage in which the suspended matter has become comminuted by pumping or otherwise.

Again, the storing of large quantities of screenings must necessarily be more objectionable than the storing of the digested sludge of a modern settling tank. Not only is the volume of this digested sludge less, but it is also of a less objectionable character.

Settling tanks have a further advantage over fine screens, in the speaker's opinion, in that with them it is possible to omit entirely the use of sludge beds, especially when settling and sterilization provide adequate purification. This can be done by creating a mechanical circulation through the settling tank. The suction pipe for this circulation system should extend down into the digested sludge and the sludge should be raised to a weir box a few inches above the water surface in the settling basin. It should be provided with an adjustable weir to regulate the discharge of digested sludge into the effluent in the proportion that such sludge bears to the volume of sewage flow. The remainder of the digested sludge thus circulated should be returned to the influent pipe, and, as it has a greater specific gravity than the fresh sewage, it will tend to entrain the suspended matter in such sewage, thus resulting in a higher degree of settling in the upper compartment of the tank. The continuous and uniform discharge of digested sludge retains at all times the maximum effective capacity of the sludge digestion chamber and prevents any sudden disturbance in its contents.

The direct discharge of digested sludge, uniformly and continuously, into a body of water with the effluent is unobjectionable from a sanitary standpoint, and will cause less shoaling in the stream than the silt transported in storm-water sewers.

The speaker does not desire to convey the impression that there is no place where fine screens might be given preference. There are many instances, especially when the sewage contains certain trade wastes, whereby the preliminary process of screening—oftentimes of fine screening—is of great value to the subsequent treatment.

In reference to the nuisance created by the composting of large quantities of solid matter removed from fresh sewage, the speaker noted an absence of seriously offensive conditions at the storage yards.

Mr. Potter.

At Dresden a great number of chickens had free run over these piles. When the screens were first put into operation, they attracted great numbers of flies during the summer, but after the introduction of the chickens there was a marked absence of flies. No one around the plant had any hesitancy about eating the eggs or the poultry raised on this diet.

Concerning the cost of operation and maintenance of fine screens, the author's data, based on German practice, must be modified, of course, to adapt it to American conditions. The compensation of German labor is less than half that in the United States. As a rule, two shifts are employed in European cities, and three shifts would be necessary in America. Furthermore, it has been the speaker's experience that, as a general rule, the attention given to such plants in Germany cannot be secured in America at any price. It is hoped that this will not always be so. As a result of neglect in attendance, unsatisfactory results are only too often obtained, for which the machinery is blamed.

Screens may be justified because screenings are better fitted as a base for fertilizers than digested sludge. At Dresden, it is proposed to put in a plant for the utilization of the screenings in this way. No less an authority than Mr. John D. Watson has recently reversed himself as to the value of sludge as a base for fertilizers.

In conclusion, the speaker desires to raise this point for discussion: has the action of the German Government in permitting the discharge of settled or screened sewage into the streams of the country been justified? The speaker believes that it has. It has brought about an immediate improvement in all rivers. It has rendered possible the solution of problems otherwise impossible because of the great expense involved in more complete processes.

It is true, of course, that in cities like Birmingham, England, where the sewage flow exceeds the stream flow, more perfect purification is mandatory, but cities thus situated are the exception, especially in America.

Mr. C. E. Gregory.

Charles E. Gregory,* Assoc. M. Am. Soc. C. E. (by letter).—Mr. Allen's paper supplies the Profession with many necessary data concerning the use of fine screens for the treatment of sewage. In the writer's opinion there is a very extensive field for such screens or other devices which will remove a considerable portion of the suspended solids from sewage which is to be discharged into harbors or rivers having sufficient volume of flow to dilute it materially. The best fine screens undoubtedly remove from fresh sewage a very considerable quantity of materials which are most potent in producing nuisances.

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* Mt. Kisco, N. Y.

Mr. C. E. Gregory.

It has been the writer's experience, in most cases where he has had to examine and correct conditions which have given rise to complaints of local nuisance along the water-front of New York Harbor, that the materials directly causing the nuisance are such as would have been removed by one of the better fine screens. River or harbor waters are capable of digesting without nuisance much greater quantities of sewage from which the coarser and heavier suspended solids have been removed than would be indicated by a comparison of the quantity removed with the total organic matter.

The prevention of the accumulation of material deposits of sewage silt by fine screening or sedimentation has prevented nuisances in practically every case where it has been tried. Large quantities of screened sewage from Dresden and Hamburg are discharged from submerged outlets directly into rivers where the dilution is not great, and apparently the discharge is without detrimental effects. London and Glasgow discharge enormous volumes of effluent, which has been treated by chemical precipitation only, into relatively small streams in which the net outward flow is exceedingly small. All the sewage from London is discharged into the Thames at two points, one on each side of the river, within a relatively short distance from each other. The enormous quantities of organic matter thus discharged reduce the dissolved oxygen content of the river to a very low point in the immediate vicinity of the outfalls, but the river water purifies itself and regains its dissolved oxygen very rapidly, both up stream and down stream from the outfall points. It is reported that the river water is practically as pure 20 miles down stream from the Crossness outfall as it is above London where it is taken for water-supply purposes. Many other instances might be cited to prove conclusively the enormous digestive capacity of waters where the heavier suspended solids have been removed from the sewage.

Recent full-sized experiments in Essen, Germany, have indicated, however, that two-story sedimentation tanks, with retention periods as short as 20 min., will remove from certain sewages about 90% of the matters removable in this way. If it is found that the retention periods for a sewage are short, and if suitable space and location are available, a short-time sedimentation tank will prove considerably more effective and less expensive than a fine screen for removing the suspended matters which have a maximum nuisance-producing effect if allowed to remain. Each particular case, however, will have to be studied and decided on its merits, taking into account:

First.—The nature of the sewage;
Second.—Available space;
Third.—Physical character of the site;
Fourth.—Cost of land;

Mr C. E. Gregory.

Fifth.—Relative facility for removing sludge or screenings without nuisance; and

Sixth.—Purity of river or harbor waters required.

Either fine screens or sedimentation tanks may become offensive and objectionable when not properly cared for. The daily removal of screenings is necessary and, if neglected, may cause serious objections in a residential neighborhood.

The lower standards of purity for rivers and harbors, which have been adopted by Germany during the last decade, when compared with corresponding standards established in England, France, and America, during the same period, seem to the writer to have been fully justified by experience, and their practice, therefore, merits adoption by the various authorities of the United States where conditions are similar.

Mr. Hering.

RUDOLPH HERING,* M. AM. SOC. C. E.—The author has given the most complete description of apparatus for screening sewage that is published to date. Heretofore, the best description was by Dr. William P. Dunbar,† but, being a physician and a sanitarian, he does not enter fully into the engineering features. Therefore Mr. Allen's account of these features of practically all the screens that have been used for the treatment of sewage is welcome.

The speaker has only one criticism to make, and that relates to the title of the paper: "Clarification of Sewage by Fine Screens". Screens do not clarify sewage at all. Even the finest screens do not remove turbidity, and a turbid sewage is not one that has been clarified.

Screens are used for the purpose of, and are of most value for, retaining floating matter. Floating matter may clog pipes or pumps, and it usually makes the surface of watercourses, into which it may be discharged, offensive.

There are in use a number of different designs, which have originated mostly in Germany. Almost every conceivable arrangement has been tried there. Most of the screens are apparently satisfactory; each engineer usually prefers his own. All have some advantages over others, and an outsider, who takes a general point of view, can usually determine the specific conditions which are best suited for one or the other design. The author has done this, and has given his general conclusions.

The general conclusions which the speaker has drawn from his study of screens, are substantially as follows: Screens which are to be used before the sewage reaches pumps may be quite coarse; they may be bar screens. It would be throwing away money to use a fine screen, like that of the Riensch-Wurl type, before a pump and for

* New York City.

† "Principles of Sewage Treatment;" First Edition, Translated by H. T. Calvert, Lond., 1908.

Mr. Hering. no other purpose, because a pump, and particularly a centrifugal pump, will pass very large solid matter without objection.

Coarse screens are also sufficient for any sewage treatment, when the sewage first goes into tanks where it is supposed to be detained. Fine screens are unnecessary in this case because coarse matter may remain in the tank. The heavy matter will settle to the bottom and the light matter will stay on top. Light matter can be removed much more thoroughly and cheaply from a tank than from a screen.

Fine screens have a different purpose than coarse screens and, therefore, a different line of application. A fine screen is needed when the raw sewage is to go out into a river. A coarse screen would not be satisfactory, because it does not prevent floating matter, visible on the surface of the water, from passing out. Take the case of New York Harbor, or along any river, large or small. If the sewage is to receive no other treatment than screening, a fine screen is necessary.

Fine screens are also desirable if the sewage is to be put through sprinkling filters, or if it is to be turned upon sand filters. By passing sewage first through the fine screens, a much greater quantity can be put on each acre, for oxidation, than is otherwise possible before clogging will result.

Parenthetically, it may be added that in some of these sewage filters even the finest screens may not prevent clogging. The speaker has pulled out from under sprinkler nozzles spreading sewage over a bed, lumps of matter, mycelium and other fungous growths, almost as large as one's fist; and yet the sewage had been fine-screened. Of course, the fungus had grown to large size where it was found, and plants will grow large, even in the sewage beds themselves. Screens do not prevent clogging from such causes, nor will they stop grease from accumulating on and obstructing the beds.

The speaker thinks that in the future there may be another application for fine screens not found in use at present. It refers to the removal of suspended matter before the sewage reaches an outfall. Quite a number of years ago the speaker was shown a result in England which then astonished him. Dr. Travis, in Hampton, having taken some sewage out of a sewer, passed it through filter paper, which made it perfectly clear. He said that this sewage was, not only clear, but on standing it would not get foul. He showed the speaker samples which had been standing for weeks and months, and they had no odor.

The speaker has found the same result in American cities. If you take sewage, as it leaves a house or soon after, and filter out all the suspended matter—in other words, get a clear solution—it very rarely becomes foul on standing.

When sewage leaves the house, nearly all matter which becomes foul is in a solid or colloidal form. There is very little organic

matter in solution: a little urine and a little of the animal and vegetable juices in the kitchen waste water will be about all. The great volume of the liquid leaving the house is water. The speaker has had sewage analyzed as it leaves the house and has found that the organic matter, the only matter with which we are concerned here, is nearly all in a suspended condition. Mr. Hering.

Now, as the sewage travels down the sewer this solid matter is broken up, comminuted, and dissolved, and the liquids leave the animal and vegetable cells, so that, at quite a distance down the sewer, one will find a large quantity of organic matter in solution. The figures that Mr. Allen has given in the tables were obtained by various authorities, and usually give the results at the mouths of the sewers. The speaker has thought that the authorities ought to add how far the sample was taken from the average point of sewage origin, because the farther the sewage flows the more organic matter it will hold in solution.

In the future, as our population increases, there is going to be more trouble or expense than now in treating sewage to obtain higher standards. It may then become more expensive than now to oxidize the stronger solutions of organic matter below the sewage outfalls. It may then be necessary to remove the suspended matter higher up and as near to its origin as practicable, because much of the organic matter which soon dissolves cannot be treated as economically in liquid as in its previous solid form.

Take, for instance, a case like the Passaic Valley Sewer, near New York City. The main sewer is more than 20 miles long. When the sewage of Paterson, at the upper end, gets to the lower end of the trunk sewer, at the pumping station, fine screening is superfluous. There is almost nothing left to screen out except the very coarse and slowly soluble matter which has come down that great distance and must be taken out simply to protect the pumps. Most of the organic matter is churned up and comminuted. It is the same as at the outfalls of the longest sewers in London, Berlin, or Paris. Most of the organic matter which could have been taken out before is still there, but is so finely comminuted that screens cannot remove it.

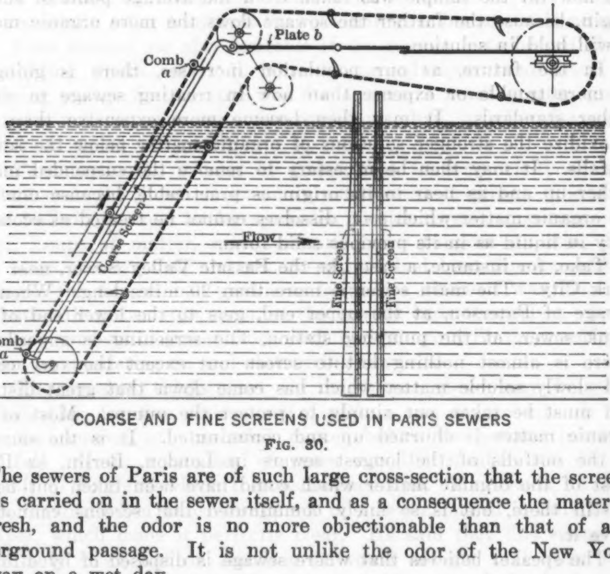
The speaker believes that where sewage is disposed of by dilution, a removal of the suspended matter nearer the origin may often be economical, and may increase the diluting capacity of the stream.

Besides screening, there is one other way in which one can quickly get most of the suspended matter out of sewage, and that is by settling basins. It is known that more suspended matter can be taken out of sewage by sedimentation than even by the usual method of fine-screening. This may not always be true, however, if the screening is done early, when more screenable matter is floating.

Mr. Hering. than later. Usually, the preference between screening and settling, and the best locations for treatment, will be determined by the question of relative cost for the advantages gained.

The speaker believes, as the author has argued, that the question of screening and its relative advantages should be studied a little more thoroughly than has been done heretofore.

Mr. Granbery. J. H. GRANBERY,* M. AM. SOC. C. E. (by letter).—The screening methods in the Paris sewers may be of interest. The process has been in use there for some time—since 1867, according to Mr. M. Ducat, Assistant Engineer, *Bureau des Eaux et des Egouts.* With the exception of changes of motive power, and improvements in mechanical details, the apparatus is practically the same as then installed, and has been giving satisfactory results.



COARSE AND FINE SCREENS USED IN PARIS SEWERS

FIG. 30.

The sewers of Paris are of such large cross-section that the screening is carried on in the sewer itself, and as a consequence the material is fresh, and the odor is no more objectionable than that of any underground passage. It is not unlike the odor of the New York subway on a wet day.

The screening is done in certain portions of the sewers, which have rectangular channels provided for this purpose, and consists of both coarse and fine screening. The device is shown diagrammatically by Fig. 30, and consists of a coarse screen composed of inclined bars about ½ in. apart, followed by one or two finer screens which are removable. A series of comb-like bars (*a*, Fig. 30), having toothed

* New York City.

parts which project between the bars of the coarse screen, are drawn upward mechanically, collect the coarse screenings, and drop them on a delivery plate, *b*. Mr. Granbery.

Portions which adhere to the combs may be swept off on to the plate by the operator before the combs again enter the channel. Nearly all the coarse suspended material is at or near the top of the liquid mass. The finer material, which is more evenly distributed, and the material which passes the coarse screen, are caught by one of the fine screens. These are used singly, but the second one is lowered preparatory to removing the one in position, when the latter is to be cleaned. The fine screens are withdrawn and cleaned by hand above the channel.

The apparatus is mounted on a car which is driven by electricity, and the whole is moved against the current while the screening is being done. This car can pull a trailer which carries a hydraulic press. The latter is used for baling the screenings, which are then taken to the "quai" (or to the railroad yards), whence they are shipped to the agricultural districts and used for fertilizer, or to low land that requires filling in. The Quai de la Megisserie, on the Seine, is one of these loading stations, and the sewer which parallels it has a wide channel in which boats are operated. These boats may be used to transfer the baled screenings along the "quais".

The channels are normally filled to within a foot of the floor, on which carts may be used in certain portions, and this floor opens on the "quai" (in the case referred to) through openings protected by steel doors. These doors have been provided since the "inundation", and are arranged to prevent the water of the Seine from backing up into the sewers, which had caused considerable trouble at that time.

These works are in the heart of Paris, and do not cause any nuisance. There is no surface indication of their presence, and one may pass the loading stations day after day for weeks without becoming aware that the traffic carried is sewage.

The liquid, after screening, is not discharged into the river, as might be supposed, but is pumped under it and discharged by gravity through closed conduits to the agricultural park at Acheres, from 25 to 40 miles distant.

In the park the underground conduits have branches which discharge, through valves operated from the surface by hand, into open main ditches, and thence into laterals which are spaced and located to suit the particular crop grown. The liquid discharged has the characteristic whitish color and appearance of dirty dishwater, and an odor that is offensive at the point of discharge but not noticeable 10 ft. from it. The flow in these open ditches is controlled by sluices.

The soil acts as a filter, and the plant growth seems to absorb all deleterious matter, for the soil does not seem to become foul. The

Mr. Granbery.

filtered liquid is collected by a series of underground drains and open canals and is claimed to be potable. It is certainly as clear and sparkling as any of the bottled spring waters sold in our cities, and its course in the river, into which these canals discharge, can be easily followed by reason of this clearness.

Mr. Pearse.

LANGDON PEARSE,* M. AM. SOC. C. E. (by letter).—The writer wishes to congratulate Mr. Allen on his very complete paper on this subject, whereby much scattered information is collected. The writer has had occasion to test screens of various mesh in the course of his investigations for the Sanitary District of Chicago, both on industrial and domestic sewages. These studies have emphasized certain points which should be observed in discussing European and United States conditions, and have pointed to the need of careful scrutiny of published figures.

What is Fine-Screening?—Screening is divided by Mr. Allen into fine and coarse. The line of demarcation is not exact, but the writer fails to see just why Mr. Allen has selected a clear opening or mesh of 0.6 in. as the line. The writer would certainly draw the line much lower, certainly at or below 0.1 in., in the light of his experiments, as well as those of Monti (quoted in the paper). Screens on small pumping stations designed for automatic service are made with about $\frac{5}{8}$ in. clear opening, and pass most of the fecal matter in American sewages, retaining only rags or coarse non-friable material.

Methods of Analysis.—The writer has long felt that the present methods of studying sewages, from a mechanical standpoint, are not complete. Chemical analyses show nothing of the size of the particles contained therein, and but little of the true strength of a sewage. Biochemical analyses, studying the dissolved oxygen consumption, give truer indications of the real or absolute strength. The mechanical indications would best be given by screening, using screens of different mesh, perhaps in a nest, like sand sieves. The data presented by Mr. Allen on the results of screening show a concordance of results with different mesh which perhaps can be explained if the character of each sewage is defined. Comparative removal by meshes of different sizes must be studied on one and the same sewage, to be conclusive. Even then it is not always easy to obtain a uniform sewage. Again, samples of both raw and screened sewage must be collected frequently, and averaged for definite periods. Correct sampling, particularly of the crude sewage, is often difficult. The writer has concluded that the best method is to collect samples of both influent and effluent and weigh all the screenings, determining the percentage of moisture. Then the dry material caught is calculated in parts per million and added to the content of suspended matter in the screened sewage to ascertain the quantity in the crude sewage. From these figures is calculated the

* Chicago, Ill.

percentage of removal. The suspended matter is determined on both crude and screened sewage—and the percentage of removal is calculated as a check. The former results, however, are usually more consistent. Mr. Pearse.

Screen Factors.—The removal of suspended matter by screening is a complex function having many variables. It depends on:

1.—The size and shape of the mesh or screen;

2.—The loss of head under which the screens are operated;

3.—The character of the suspended matter;

4.—The freshness of the sewage, which in one of domestic origin will be influenced by the area sewered, the time of concentration, and the condition of the sewers;

5.—The dilution of the sewage; *i. e.*, the gallons of flow per capita, or the concentration of suspended matter;

6.—The type of screen.

In the writer's opinion, Table 11 would be increased in value many fold if columns were added giving (1) the area tributary to the screens; (2) the concentration of population per acre; (3) the concentration time of the sewage; (4) the flow, in gallons per capita; (5) the minimum and maximum opening, in inches; (6) the percentage of gross area which is opening; and (7) the content of suspended matter, in parts per million.

Foreign vs. American Sewage.—The great difference in American and European conditions might be shown by the foregoing data, particularly by the suspended matter content. For instance, by referring to Table 10, it will be seen that the Dresden sewage contains 371 parts per million of suspended matter. Many American sewages are more dilute. Industrial wastes are often more concentrated. As a rule, tannery, stockyards, and Packingtown wastes around Chicago seldom exceed 7 500 parts per million, and are usually greater than 500 parts per million. Domestic sewages in the United States seldom exceed 250 parts per million. The average suspended matter at the Thirty-ninth Street Pumping Station during 3 years has been 144 parts per million, from daily samples of twenty-four portions.

Dr. William Philipps Dunbar* gives the following characteristics of German sewages:

Place.	Suspended matter, in parts per million.	Free ammonia as NH_3.
Hamburg	299.4	27.5
Frankfurt	1 390	31.5
Wiesbaden	74	37

* "Principles of Sewage Treatment," First Edition, Translated by H. T. Calvert, p. 34.

Mr. Pearse.

The writer does not believe that Mr. Allen has cited sufficient American data to maintain that the efficiency of fine-screening is equivalent to settling, particularly if fine-screening be defined as 0.6 in. or less for the clear net opening. Of the results given in Table 11, all but three are foreign data, on conditions possibly widely different from those obtaining in the United States. Of the three results in the United States, the Jennings screen is on a peculiar sewage, direct from the stockyard pens, containing much manure, hay, straw, etc. The average content of suspended matter is 340 parts per million. On tests made by the Sanitary District, a removal of 33% was obtained (based on "computed" influent). A 12-hour period of settling removed 64 per cent. Of the other two places quoted, Reading and Brockton, the results at Reading are placed much lower by George W. Fuller, M. Am. Soc. C. E.,* where 15 to 20% removal is noted, instead of 42% as in Table 11. This discrepancy is discussed in the following paragraph more at length.

At Brockton a very concentrated sewage is handled from a separate system. The data in Table 7 are quite inconsistent with the text which follows. From Table 7 the removal of suspended solids is 983 — 282 = 701 parts per million. From the text, in the same interval, 1.4 tons of screenings per million gallons were removed, or 336 parts per million, if the 1.4 tons is dry weight. This would give an efficiency of 54% removal instead of 71.3 per cent. If the weight is wet, the percentage would be much lower. Probably both figures are too high. It would be of interest if Mr. Allen would state the source of the analyses, and explain whether they are more than an average of twelve samples, collected during a few minutes once a month. Judging from reports of the Massachusetts State Board of Health, a removal of 983 parts per million of suspended matter does not represent average conditions, but is extremely high. In 1911, the yearly average was reported as 363 parts per million.

Reading Screen.—The data quoted from Emil Kuichling, M. Am. Soc. C. E., and from "Sewage Sludge" by Elsner, Spillner, and Allen, are not consistent. Mr. Kuichling estimates the dried matter at 750 lb. per million gallons, or 90 parts per million, giving an efficiency of 42% on a sewage containing a total of 215 parts per million of suspended matter. Mr. Allen's figures agree with a statement on page 77 of the Annual Report of the City Engineer of Reading (1910), where 31 cu. ft. per million gallons of screenings were obtained with 90% moisture. Taking the weight at 63 lb. per cu. ft.,† 1 953 lb. of wet screenings would be recovered, or 195 lb. of dry screenings per million gallons. This would be equivalent to 23.4 parts per million. Even on a weight of 70 lb. per cu. ft. the equivalent removal

* "Sewage Disposal," p. 381.

† "Sewage Sludge," p. 208.

is only 26 parts per million. From the City Engineer's report for 1909, the suspended matter in the screened sewage appears to average 151 parts per million. Hence the screening efficiency is $\frac{26}{177}$, or about 14.7 per cent. Mr. Pearse.

The Reading screen was out of service the equivalent of 63 days in 1910, and 77 days in 1911. This would point to the need of duplicate installations.

Scrutiny of Published Data.—It is evident, from a comparison of the data quoted regarding the three American plants, that the efficiencies of screens are probably lower than Mr. Allen's figures. More detailed figures are needed on long-continued tests in order to establish the true figures. If a settling basin is to be rated by its average accomplishment, so should a screen. The writer, therefore, thinks that Mr. Jennings' figure of 63%, based on a peak load, is not a fair one to compare with an average load on a settling basin.

Other Types.—Although not strictly for sewage, two classes of screens may be added, to make Mr. Allen's references complete. One is the rotary or drum screen devised by the United States Reclamation Service.* This was for the purpose of removing trash from an irrigation channel on the Salt River project, and was provided with ½-in. mesh. The other is the type of machinery known in the paper trade as save-alls. These were devised to retain particles of pulp, paper, etc., which would otherwise be carried into a stream or sewer with the wash-water. They are largely of the drum type, fed inside, like the Weand screen, but are sometimes built so that the flow is from the outside inward. Cylindrical screens of this type have been in operation for nearly 20 years. Meshes as fine as 100 per lin. in. are used. They are cleaned by water or by air.

Comparative Tests.—Comparative screening tests on the Thirty-ninth Street sewage are shown by Figs. 31 and 32. This is a domestic sewage, very dilute, containing about 140 parts per million of suspended matter, from a drainage area of 22 sq. miles, with a concentration time of about 5 hours. The dry flow is roughly 200 gal. per capita. The population tributary in 1910 was 274 000, or 19.1 per acre. The tests were all made to a constant loss of head of 0.5 ft. on wire mesh screens, 4.2 sq. ft. in area, having the mechanical characteristics stated in Table 23.

Screening tests made with screens of the same mesh, on the Center Avenue sewage, give the results noted by Fig. 31. These were made with a loss of head of 4.5 ft. on a 21 by 24-in. screen. The Center Avenue sewage contains much stockyards and Packingtown waste, with quantities of coarse material, such as paunch manure, hay, particles of meat, hair, etc. The drainage area is 660 acres, with a con-

* Described in *Transactions*, Am. Soc. C. E., Vol. LX, pp. 337–338.

Mr. Pearse. centration time of about 30 min., a day flow of 700 gal. per capita (29.0 cu. ft. per sec.), and a night flow of 400 gal. per capita (16.4 cu. ft. per sec.). The removal is far higher than at Thirty-ninth Street.

Effect on Organic Content.—The writer has been much interested in the data presented relative to the effect of screening on the organic

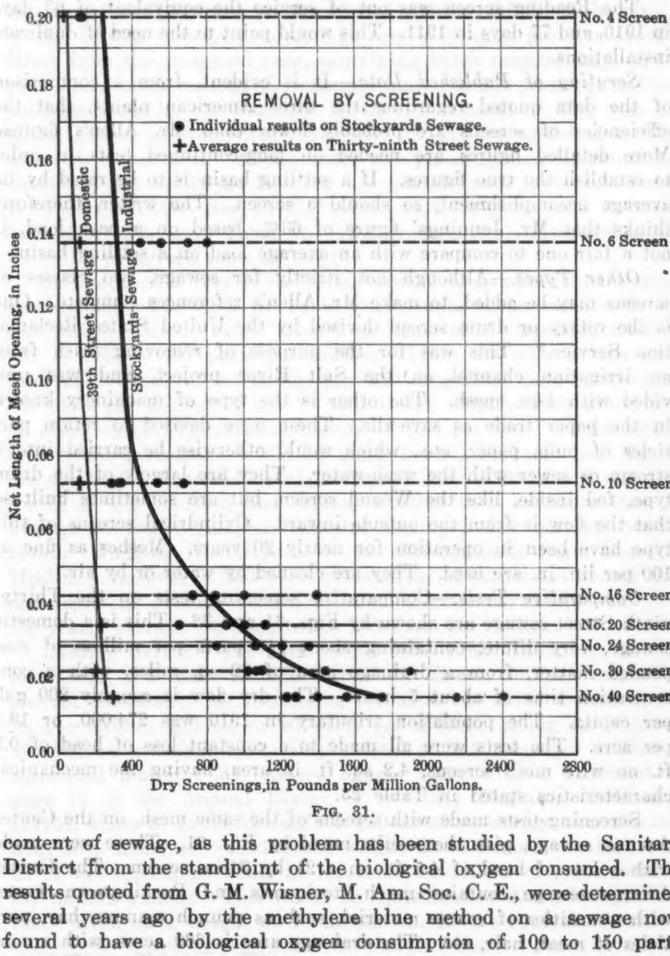


FIG. 31.

content of sewage, as this problem has been studied by the Sanitary District from the standpoint of the biological oxygen consumed. The results quoted from G. M. Wisner, M. Am. Soc. C. E., were determined several years ago by the methylene blue method on a sewage now found to have a biological oxygen consumption of 100 to 150 parts per million. Recent extended tests on the improvement made by a

30-mesh Weand type of screen on the Center Avenue sewage, with a biological oxygen consumption of 1 100 parts per million, show an improvement of only 6%, with a removal of 8% of suspended matter, from a sewage containing about 487 parts per million of suspended matter. Settling the screened sewage removed 283 parts per million of the suspended matter, making a total removal of 66% on the crude sewage, with a reduction in oxygen demand of 42% in a number of tests. This certainly does not indicate that screening is comparable to sedimentation. Mr. Pearse.

TABLE 23.—PROPERTIES OF FINE SCREENS.

Meshes per linear inch, nominal.	Diameter of wire, in inches.	Net Opening. Length of Side.		Open space. Percentage of gross area.
		Inches.	Millimeters.	
4	0.048	0.198	5.04	65
6	0.034	0.137	3.48	64
10	0.026	0.072	1.83	54
16	0.021	0.042	1.07	42
20	0.016	0.034	0.86	46
24	0.015	0.029	0.74	42
30	0.012	0.022	0.56	41
40	0.010	0.015	0.38	28

Recovery.—The screenings from the rotary screens, on a packing-house and on the Center Avenue sewer, averaged 84.3 and 86.4% of moisture, respectively, as compared with 91.5% of moisture in the sludge from an experimental Emscher tank 17 ft. deep. The organic content was high, but the fat content was not great, largely because the fats are in a finely divided emulsion, and require chilling before congelation and rising to the surface. The writer is not very confident in general of obtaining valuable recoveries from domestic sewage. Doubtless, too, the material would be open to the criticism that the foreign material, rags, etc., in the screenings would create suspicion in the mind of a farmer purchasing it as a fertilizer.

Cleaning.—The writer has always favored the use of compressed air instead of water for cleaning rotary screens of the drum type. In a screen tested in Packingtown, of a partly submerged drum type, so much water was used that a settling basin had to be built to settle the screenings. With the Jennings screen, the heavy air pressures first used were too severe for the screen fabric, but later reduced pressures have appeared favorable. The cleaning must be adequate to keep down the loss of head or clogging of the screen below defined limits; little is gained in removal after a screen once begins to clog, as the rate of clogging increases rapidly.

Mr. Pearse. *Disintegration.*—Mr. Allen refers to the disintegration of solids, with consequent passing through a screen. This would appear to be dependent on their character. Non-friable materials certainly would not break down. Further investigation is required. If pumping is needed, the disintegration produced thereby will probably exceed

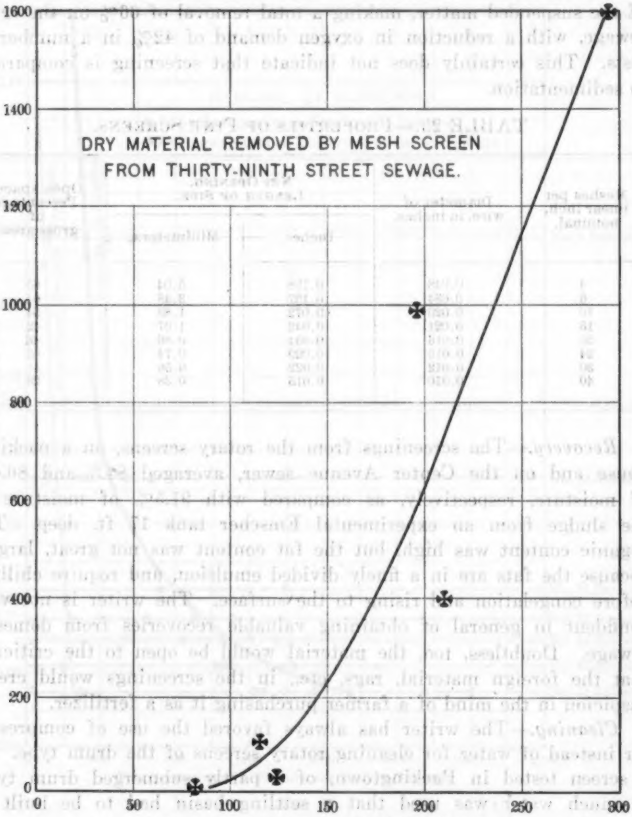


FIG. 32.

that caused by the screen. The writer has heard comment by other observers on this point, but would judge that that condition, particularly with the Riensch-Wurl type, is not continuous.

Mechanical Details.—The Weand screen requires considerable loss of head, and consequent pumping cost, in many localities. The

Riensch-Wurl and other partly submerged foreign types require but slight loss of head, compared with the Weand. Mr. Pearse.

Of the screens now in use in America, some mechanical improvement is required to put them in a permanent class. The screen at Reading may have been experimental, but it required costly repairs. The screen at Brockton has had some difficulties, as outlined by Mr. Allen. Foreign practice has developed certain types of screens which appear to have been operated with slight mechanical troubles, if any. Doubtless some can be adapted to American conditions. The Jennings screen is very simple, and is capable of being developed to a higher pitch of mechanical construction, after the experimental stage has been passed.

Specifications.—The writer thinks that, for the purchase of screens, the specifications should be drawn to secure a thoroughly satisfactory mechanism with sufficient stability and generous detail to stand the constant wear and tear, particularly if finances preclude duplicate machinery. The removal expected may or may not be a matter of contract. If it is to be guaranteed, it should certainly be defined more rigorously than in a specification recently brought to the writer's notice, in which a removal "equivalent to that of a settling basin" was required.

Results Obtained.—From a study of screening, under working conditions, on industrial wastes, the writer is inclined to believe that from 20 to 35% removal of suspended matter can be obtained with from 30 to 40-mesh screens, under favorable conditions, on certain industrial wastes. He considers, however, the figure of 30% removal, advanced by Mr. Allen, far too high for ordinary domestic sewage, particularly if from a sewer in a combined system with a long concentration time, fed by a large area in which the population uses much water.

Applications.—The writer does not share the enthusiasm expressed by Mr. Allen in his conclusions. Undoubtedly, the use of fine screens for sewage deserves more serious consideration than heretofore, but the results to be obtained are by no means demonstrated as yet. Careful study and skilled tests are required before the art of screening will be placed on as accurate and satisfactory a basis as sedimentation is to-day. Comparative results on the same sewage do not indicate that screening, even with a 30-mesh screen, is the equivalent of thorough sedimentation, nor within 50 per cent.

Mr. Allen is right in calling for more fundamental data on efficiencies and costs, and such data will be required before screening will develop widely in the United States. European data are of little value as a general guide to the results to be obtained in the more dilute sewages common in America.

Mr. Pearse. The writer is of the opinion that the application of screening to industrial wastes is a field which has been overlooked. In many industries wastes of a coarse nature are discharged, which do not readily disintegrate. The application of screening in Packingtown, particularly on paunch manure overflows, and in the stockyards, has demonstrated worth. Possibly in tanneries, wool washing plants, and the like, a use may be found under favorable conditions.

In the paper and pulp industry considerable application has already been made to retain pulp which would float away in wash-water. A screen of the drum type has also been tried at Anaheim, Cal., on the waste of a beet sugar works by C. G. Hyde, M. Am. Soc. C. E., with considerable success.

Acknowledgment.—The writer trusts that Mr. Allen will pardon this frank analysis of the situation, as it is thought that it will clear the atmosphere for the future of screening, and encourage the operators of screens to obtain really valuable data. The writer wishes to acknowledge the courtesy of the Trustees of the Sanitary District of Chicago and the Chief Engineer, Mr. George M. Wisner, for material drawn from a forthcoming report.

Mr. Whittemore. L. C. WHITTEMORE,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read with great interest Mr. Allen's presentation of available data on a branch of sewage treatment practice as yet little developed in the United States. Extended comment is made by Langdon Pearse, M. Am. Soc. C. E., particularly on the removal efficiencies and reduction in oxygen demand to be expected, as shown by extensive experiments made under his direction for the Sanitary District of Chicago. The limitations of screening in American practice are discussed at length. The writer, therefore, will discuss chiefly two points, wherein fine-screening is likely to be of value in the treatment of packing-house and stockyards wastes, in spite of its obvious limitations as compared with sedimentation. Both have been noticed by Mr. Allen, and are: (1) the possible reduction in rate of sludge accumulation in settling tanks, and (2) the partial or complete prevention of scum formation. The general character of the packing-house sewage is outlined by Mr. Pearse.

With regard to the first factor, extended investigations have shown that sludge at the rate of 8 to 10 cu. yd. per million gallons may be expected as a result of plain settling in tanks of the Emscher and Dortmund types, based on uniform flow. If corrected to allow for operation at a variable flow, to follow the discharge from the sewer throughout the day, they should probably be increased by 25% or more. With a 30-mesh rotary screen, operating between the hours of 8 A. M. and 4 P. M., an average removal of 950 lb. of dry material per million

* Chicago, Ill.

Mr. Whittemore.

gallons was obtained. Extending the run to 11 P. M. reduced the average rate to from 320 to about 500 lb. per million gallons. Reduced to sludge of 90% moisture (tank sludges have usually run somewhat higher than this figure), this is equivalent to from 2.0 to 5.5 cu. yd. per million gallons, approximately. Practically nothing was retained between 11 P. M. and 8 A. M., and some reduction in the average removal for the entire day would be necessary. On the other hand, the highest removal occurs during the hours when the flow in the sewer is greatest, and it is evident that material reduction in the rate of sludge accumulation would result with fine-screening. The saving in sludge storage capacity and area required for drying beds, if the sludge is disposed of in this manner, is obvious. A substantial saving in cost of tanks and drying beds should therefore result. Moreover, the screenings are readily handled after slight draining, whereas the tank sludges are liquid, and require several days for drying with the well-digested Emscher product, and considerably longer for other sludges, under favorable conditions. With further removal of moisture, perhaps by pressing, the screenings could probably be disposed of by incineration, as the calorific value of the dry material exceeds 9 000 B. t. u. per lb.

The second point in favor of screening is the prevention of scum formation. Scum has been continuously present in considerable quantities on the various experimental tanks, except where chemical precipitation was used. The high temperature of the sewage (70 to 90° Fahr.) and the rich organic content stimulate vigorous bacterial action, and this, with the high content of grease and light fibrous matter, promotes the rapid accumulation of scum. Comparative experiments in a Dortmund tank in May and June, 1914, under identical conditions (a 3-hour detention period), except that screened sewage was used during the hours of heavy flow in May and raw sewage in June, showed rates of scum accumulation of 0.5 and 3.1 cu. yd. per million gallons, respectively. Scum did not appear at all until the last few days in May, but was continuously present in June, reappearing very shortly after being removed. The difference is striking, and as the scum is neither liquid enough to flow readily, nor sufficiently solid to be easily handled, its removal would entail considerable difficulty in a large installation. The value of scum prevention is very apparent.

The writer fully realizes that his discussion is perhaps rather limited in its application, as the sewage dealt with is of unusual character, but it is hoped that a possible field of usefulness for fine-screening, as an adjunct to, rather than as a substitute for, sedimentation, in treating wastes of this nature, has been demonstrated.

In conclusion, the writer would suggest that percentage efficiencies of removal be scrutinized with care, and the conditions under which

Mr. Whittemore. they are determined be critically examined before too much credence is given to unusually high removals. A few scattered determinations may lead to an entirely erroneous opinion when based on analysis of small influent and effluent samples by the usual methods. As illustrative of this, it may be of interest to note that, during a 27-day run with the experimental rotary screen, daily efficiencies ranging from an apparent increase of 7% to a reduction of 53% and averaging 32% were found. The screen was operated for 8 hours each day, and influent and effluent samples made up of portions collected every half hour were analyzed by the Gooch crucible method. The rate of flow was constant, and all conditions were particularly favorable for securing accurate results, yet wide differences are noted, and the apparent increase is of course impossible, as a matter of fact. Based on the method of computing the influent analyses, as mentioned by Mr. Pearse, results ranging between 12 and 23% were secured, which are in all probability far nearer the truth. Although the effluent may be affected by errors of sampling and analysis, as well as the influent to a certain extent, it is possible that, when the coarse particles not adequately represented in the analysis are eliminated, the effluent analyses are on the whole more representative, and this method is more trustworthy where the flow and weight of screenings can be ascertained accurately. In relation to this matter, the writer would express the opinion that calculating the efficiencies of screens to tenths or even hundredths of 1% is entirely unwarranted, when consideration is given to the accuracy of the data on which they must necessarily depend. The nearest percentage is undoubtedly well within the limit of error, and is entirely sufficient for practical purposes, without giving a false idea as to the accuracy of results.

Mr. Greeley. SAMUEL A. GREELEY,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has been much interested in this comprehensive paper. He inspected the screening plants at Hamburg and Frankfort in June, 1908, and again in September, 1911, at which time he also examined the screening plant at Dresden, then in its first year of operation. It has been interesting to compare the data secured during these inspections with those given by Mr. Allen.

Hamburg.—The River Elbe divides the city into a larger northern section and a smaller southern section, for each of which there is a screening plant. The older plant described by Mr. Allen is on the river-front, in a very busy, thickly built-up section of the city. Within four blocks are the seaman's school and the entrance to the new tunnel under the river.

The time of flow of sewage to the plant was estimated to be from 2 to 4 hours, depending on the volume. The sewage appeared to be fresh and contained a strikingly large proportion of unbroken feces.

* Winnetka, Ill.

Mr. Greeley.

At the time of the writer's visit in 1911, it was stated that the volume of sewage flowing through the screens was about 50 000 000 gal. per 24 hours. The screens removed from 13 to 65 cu. yd. daily, the average removal being 26 cu. yd. of wet screenings per 24 hours. Mr. Allen states that the removal is 18 cu. yd. per day.

The sludge and screenings are dumped into covered steel barges which are towed to dumping grounds near Neuenfelde, a farming locality along the Elbe, some miles below Hamburg. The tank boats are taken to an unloading station, on a small creek. The station has an inlet for docking the boats, an unloading crane, some small trucks, and a narrow-gauge movable track. The crane lifts the body from the truck to the boat. There it is filled with a shovel, and then replaced on the truck, and run out on the dump. At the time of the writer's visit, the dump was about 5 ft. deep and 75 by 50 ft. in area. It was very foul, giving off a strong pig-pen odor. The material was carted away to the fields in farm wagons.

The country about this unloading station was devoted to truck gardens and orchards, through which the writer walked about 2 miles before coming to the unloading station, and throughout most of this distance the pig-pen odor from the screenings was clearly evident.

Frankfort.—The sewerage of Frankfort is on the combined system. The sewage from the whole city is treated at one disposal plant by screening and sedimentation. As is common in German cities, the sewers are well built and have good slopes, and the sewage comes to the plant in a fairly fresh condition. However, it did not appear to be as fresh as that at Hamburg and Dresden.

The average volume flowing to the treatment plant is about 26 000 000 gal. per 24 hours. The screens remove from 20 to 26 cu. yd. of wet screenings per 24 hours.

The screenings are taken with the sludge from the settling basin to centrifugal drying machines, and the dried material is burned with the city refuse in incinerators. The centrifugal machines are in a large building, and the dried material is handled on conveyors. The odors about this part of the plant were very much like those in a garbage reduction building.

It is interesting to note that, up to the latter part of 1909, the sludge and screenings from the Frankfort plant were disposed of on land. This method was unsatisfactory, and the present more costly method of disposal was adopted.

Dresden.—Dresden is sewered on the combined plan. The superintendent at the screening plant informed the writer that the time of flow of sewage from the center of the town to the screening plant was only about 0.5 hour, and for most of the sewage, did not exceed 2.25 hours.

Mr. Greeley. The average quantity of sewage flowing to the plant was 26 500 000 gal. per 24 hours. During storms, this quantity is greatly increased. The superintendent stated that the revolving screens removed from 13 to 16 cu. yd. of wet screenings per 24 hours.

At the time of the writer's visit only one of the disk screens was in operation, and the difference in level of the sewage on the two sides was 11.8 in. The maximum loss of head expected was 23.6 in. The writer was told that the disk screens sometimes required washing with a hose once every hour to remove fat and grease. At other times, one washing on each 8-hour shift was sufficient. One man attended to the ordinary operation of all four screens.

The screenings were taken to long wooden troughs in the yard. These troughs had false bottoms of wood set about 18 in. above the paved floor of the yard. The liquid from the screenings drained to the floor and flowed back in a sewer to the screens. The troughs were roofed over, but not housed in. The dry screenings are said to be good for fertilizing, and are removed by farmers, who pay for the privilege. Although there was only a small accumulation at the plant when seen by the writer, the material was offensive, having the characteristic pig-pen odor.

Percentage of Removal.—It is hard to express the removal of suspended matter by screens as a percentage of the total original suspended matter, because of the difficulty of securing representative samples for analysis. The data on removal, as obtained by the writer, and as given by Mr. Allen, are summarized in Table 24, in which the percentage of removal is shown. This "percentage of removal" is computed from the weight of screenings caught, the percentage of moisture contained in the screenings, and the suspended matter in the crude sewage. The dry material in the screenings, in parts per million, is calculated against the total suspended matter in the crude sewage. Screenings are assumed to weigh 60 lb. per cu. ft.

The percentage of removal at Frankfort, calculated from the content of suspended matter in the crude and screened sewage, is 10 per cent.*

TABLE 24.—PERCENTAGE OF REMOVAL OF TOTAL SUSPENDED MATTER BY FINE SCREENS.

Plant.	Undrained Screenings, in Cubic Yards per Million Gallons.		Suspended matter in crude sewage, in parts per million.	Percentage of moisture in screenings.	Percentage of Removal:	
	Greeley.	Allen.			Greeley.	Allen.
Hamburg........	0.52	0.34	300	87	4.4	2.9
Frankfort........	0.88	2.48	411	85	6.2	17.5
Dresden..........	0.55	0.97	300	84	5.7	10.1

* *Die Frankfurter Kläranlage.*

The sewages represented in Table 24 are more concentrated and fresher than is usual in American cities. Mr. Greeley.

Disposal.—Mr. Allen correctly calls attention to the importance of the disposal of screenings when he states that "another and more probable cause for offense lies in the disposal of the screenings". The writer does not consider that Mr. Allen has given sufficient consideration to this difficult problem. The screenings at the plants visited were very unsightly and unpleasant to handle, quickly fouling the cars, conveyors, and chutes, and causing offensive odors. Certainly, when placed on land, the screenings become very foul. Disposal by incineration confined the odors to within the works, but added to the cost of disposal. As the efficiency of screens increases and greater quantities are removed, the disposal problem also increases. When comparing fine-screening with sedimentation, the matter of disposal must be considered.

Summary.—Judging from his investigations, the writer considers that a removal of suspended matter from domestic sewage amounting to 30% is higher than can be expected in ordinary operation and under reasonable limitations of loss of head, cost, present available devices, etc. The available data indicate that a removal of 15% has seldom been reached in practice. The writer's observations lead to the conclusion that the disposal of screenings is attended with the production of odors which may create nuisances, and that adequate methods of disposal of screenings, with reference to cost and efficiency and the location of the place of disposal, are essential parts of an installation of fine screens.

E. KUICHLING,* M. AM. SOC. C. E.—The thanks of all engineers interested in the treatment of sewage are due to the author for his masterly presentation of useful data on the construction and effect of fine screens. It is the first appearance of such comprehensive information in English technical literature, as the subject has received little attention from English and American engineers until very recently. In Germany, on the other hand, for more than twenty years past, much scientific study has been given to the screening of sewage, with the view of avoiding the great expense of clarification by means of tanks and filters, in places where the effluent could be discharged into relatively large streams without causing appreciable offense; and hence most of the available data found previously were in German publications. Mr. Kuichling.

It is claimed that screens with openings from 0.10 to 0.25 in. wide will intercept a large proportion of the floating and suspended solids in sewage, together with the multitude of bacteria adhering thereto and enclosed therein, and that the effluent will consequently be

* Mr. Kuichling died on November 9th, 1914. This discussion was the last work he did.

Mr. Kuichling.

greatly improved in appearance and rendered more easily capable of rapid assimilation by the water of the stream into which it is discharged; also that the removal of such solids will greatly facilitate every other mode of further treatment. Much of this intercepted matter consists of practically non-putrescible substances like paper, cloth, cordage, leather, hair, bone, wood, bark, husks, corks, bread, fat, etc., and the remainder is mostly fresh and dried vegetable matter, such as leaves, stems, parings, rinds, and seeds, together with fragments of meat, fish, membranes, entrails, etc. These latter substances are putrescible, but usually reach the screen in odorless condition. The screenings, therefore, are free from nauseating odors and can be handled inoffensively; they must, however, be removed at frequent intervals and with the least possible comminution, in order to avoid loss of head and defilement of the effluent. To realize these claims, careful design and operation of screens are imperative.

In regard to the quantity of solid matter removed by fine screens, it may be noted that the figures in Table 11 show a range of 0.34 to 5.94 cu. yd. per million gallons, with an average of 1.90 cu. yd., and a range of 0.014 to 0.282 cu. yd. per 1 000 of population daily, with an average of 0.093 cu. yd. Presumably, the reference is to wet material weighing about 1 500 lb. per cu. yd., and containing about 75% of moisture, so that 1 cu. yd. of wet screenings represents about 375 lb. of dry substance. On this basis the aforesaid averages will be 712.5 lb. of dry matter removed per million gallons of sewage, or 85.43 parts by weight per million, and 34.88 lb. per 1 000 persons daily. These quantities are certainly large enough to make a marked improvement in the quality of the sewage, and to reduce materially the volume of troublesome sludge produced by subsequent sedimentation. It can also be inferred that the sludge deposited after the sewage has passed through a fine screen will be much more easily and extensively decomposed by bacterial action than the sludge of unscreened sewage, but comparative experiments in this direction are still lacking.

To compare the weight of the dry solids contained in the sedimented sludge of unscreened sewage with that of the screenings, it can be assumed that, on the average, the volume of such sludge is 2 000 gal. per million gallons of sewage; also that the weight of this sludge is 66⅔ lb. per cu. ft., and that it contains 85% of water, so that each cubic foot contains 10 lb. of dry substance. The volume of sludge is thus 267.36 cu. ft. per million gallons of sewage, and hence it contains 2 673.6 lb. of dry matter, while the screenings contain on the average 712.5 lb., or 26.65% of the former. It may be remarked, in this connection, that the plain sedimentation of the sewage of Providence, R. I., during 1912 yielded an average of 1 758 gal. of wet sludge per million gallons, with 9.97% of dry solids; and, if it is assumed that the wet sludge weighs 65 lb. per cu. ft. and con-

tains 6.5 lb. of dry substance, then the said volume of sludge contained 1 527.5 lb. of dry matter per million gallons of sewage. As a fine screen was not used, no further comparison can be made in this case, but, if the aforesaid average of 712.5 lb. of dry matter per million gallons could here be removed by fine screens, the latter would remove 46.65% of that which was removed by plain sedimentation. Mr. Kuichling.

The percentage of suspended matter intercepted by a fine screen varies much in different hours of the day, different days of the week, and different months of the year. It is governed to a considerable extent by the nature of the industrial wastes admitted into the sewers, and by the intensity of the rainfall which washes the streets of the city. If there are many shade-trees, and also if the streets are not frequently cleaned, a heavy storm will cause large quantities of leaves, paper, and other débris to enter the sewers in a short period of time, and will also flush out previous deposits in the sewers, thereby greatly increasing the suspended and floating solids. The variation in quantity of matter intercepted by fine screens having meshes or orifices from 0.06 to 0.08 in. in diameter is shown clearly by Metzger's experiments in 1906 and 1910 at Bromberg, Germany.* In the first experiment, with a wire-cloth screen of 0.08 in. mesh, during an entire dry day in 1906, the volume of sewage screened was 1 622 300 U. S. gal., and the weight of the wet material arrested was 8 118.66 lb., thus making an average of 338.28 lb. per hour from an average flow of sewage of 67 596 gal. per hour; and the maximum weight in one hour was 1 334.66 lb., while the minimum weight in one hour was 8.60 lb. In 1910 the second experiment was with a drum screen of thin perforated metal having circular orifices 0.06 in. in diameter; and, during one week, 77 432 lb. of wet screenings were intercepted from 8 402 773 U. S. gal. of sewage, the averages thus being 11 061.7 lb. from 1 200 396 gal. per day; the maximum in one hour was 1 750.45 lb. from 71 064.4 gal. of sewage, and the minimum was so small that the movement of the screen could be stopped from 1 to 7 A. M. The wet screenings were deposited in a receptacle and weighed, after decanting the supernatant water, and it was found that the thick pulpy matter left contained 34.3% of moisture. On this basis the aforesaid rates of removal will become an average of 6 054.2 lb. of dry solid matter per million gallons per day, and a maximum of 16 183.1 lb. per million gallons in one hour; and by reducing to parts by weight per million, these figures become, respectively, 724.2 and 1 935.8. The cost of operating this screen for a week was at the rate of $2.45 per million gallons, including all expenses for attendance, oil, waste, air compressor, lighting, and electric power, and it is claimed that this rate will be materially reduced in regular operation.

* Described in *Technisches Gemeindeblatt*, Vols. 9 and 13.

Mr. Kuichling. Data like the foregoing, together with those presented by the author, prove conclusively that fine screens are highly efficient and economical in preparing sewage for admission into comparatively large streams where the dilution of the effluent is permissible; and they also indicate that such devices will prove advantageous in cases where a high degree of purification is required, by relieving settling tanks and filters of a large and undesirable burden. It is to be hoped that comparative experiments will soon be made with the same sewage, which will demonstrate the effect of fine-screening on different modes of sewage treatment.

The chief difficulty in screening sewage is in preventing serious obstruction of the orifices, and in cleaning the apparatus in an unobjectionable manner. Many ingenious devices have been invented to keep the submerged portion of stationary gratings clear, and to remove above water the material intercepted by movable screens. In the former class a considerable quantity of suspended matter is unavoidably comminuted and mixed with the effluent, and in both classes the wet mass must be removed from the moving parts after emerging from the sewage. This removal has hitherto been accomplished by combs, scrapers, brushes, and jets of water, steam, and compressed air; but in all such operations some spattering takes place, whereby finely-divided sewage matter gets into the surrounding air which must be breathed by the attendants, and may thus cause sickness. This danger can be obviated by applying a suction apparatus to the wet screenings and removing them in a pipe to a receptacle, without producing spray and polluting the air. Such a device, however, has not yet been developed, probably because an active demand therefor does not exist.

The speaker became interested in cleaning sewage screens by the suction or vacuum method about 9 years ago, and laid the matter before several makers of vacuum cleaners, as well as a number of skillful mechanical engineers. To his astonishment, these men asserted that wet pulpy material could not be removed from a screen in this way, as the suction pipe would soon become clogged. They did not know that many years ago this process was applied successfully to the cleaning of cesspools and privy vaults, and is still in use in unsewered communities; nor that a distinguished Dutch engineer, Capt. Charles T. Liernur, about 40 years ago, applied it to the removal of excreta through an extensive system of cast-iron pipes laid under the streets of Dordrecht and several other small cities, where the matter was delivered by suction to a central station and there converted by evaporation into a commercial fertilizer. These plants were in operation for many years, and are probably still in use; hence the practicability of the method can be regarded as established.

The designing of fine screens for sewage is essentially a problem

for mechanical engineers, but as these devices are used only by civil or municipal engineers, a knowledge of the numerous details involved becomes necessary to the latter. It is not an easy matter to compute the stresses in an inclined disk 30 ft. in diameter, like the Riensch screens at Dresden, subjected to the load of wet screenings and possibly a considerable hydrostatic pressure on a part of its surface; yet the clear recognition of these forces is necessary in order to secure durability and correct action. Means must also be provided for varying automatically the speed of the moving parts, so as to conform to the different hourly quantities of suspended matter intercepted, otherwise there will be either a waste of power or an excessive loss of head by obstruction of the orifices. Mr. Kuichling.

Reference may also be made to the loss of head involved in the passage of sewage through a fine screen. The author has cited (page 930) a coefficient of discharge of 0.4 for the small slots of a Riensch screen, on the assumption that from one-half to one-third of their area is obstructed. It would be interesting to know how this value was reached, as a much larger figure is commonly used in hydraulic computations. Small orifices and narrow slots in a thin plate of metal have a coefficient of discharge of 0.63 to 0.65, but its value may perhaps become less if the orifice is very irregular in shape. Experiments in this direction are lacking, and few observations of the actual loss of head at a partly clogged screen have been recorded. Estimates of 1 ft. loss of head are occasionally found, whereas at Hamburg the actual loss ranges from 8 to 30 in. It is obvious that this loss might be utilized to control the speed of the moving screen, in order to prevent excessive back-water and the development of internal pressure in a closed conduit.

Of the various fine screens described in the paper, the one invented by Mr. Riensch and manufactured by Mr. Wurl seems to have met with general approval. It has the advantage of presenting a large area to the sewage, thus attaining a minimum loss of head; it is also easily cleaned, and when built without a submerged bearing, as indicated by Mr. D'Olier, it can readily be lifted from the sewage for convenient examination or repair; the cleaning is done below the level of the floor used by the attendants, and the revolving brushes can be covered with a hood to prevent the escape of spray and particles of solid matter into the atmosphere; there is no appreciable pressing of soft solids through the slots or orifices by the brushes, and the drippings from the intercepted matter before its removal fall only a short distance into the effluent, thereby reducing the evolution of dissolved gases and odors from the liquid by prolonged exposure and agitation.

All these features must be taken into consideration in designing a fine screen for sewage, both in order to secure efficiency and to

Mr. Kuichling. render the work of attendance as healthful as possible. At its best, such work is unpleasant, and much more will be accomplished if it is made free from the danger of contracting disease. Economy of construction and operation is the final consideration in a successful design.

Mr. Hammond. GEORGE T. HAMMOND,* M. AM. SOC. C. E. (by letter).—This paper, on a timely subject, is of unusual interest in that it gives, in condensed form, a very complete treatment of the subject. The writer believes that the fine screen has a large field of usefulness, both as the sole method of sewage treatment under certain conditions, and also as an essential part of methods which aim to secure a higher degree of purification. While making studies and investigations, preparatory to the design of an experimental sewage disposal plant for the Borough of Brooklyn, the writer accompanied Rudolph Hering and E. J. Fort, Members, Am. Soc. C. E., through England, and Mr. Fort through Germany, for the purpose of observing and noting the state of the art of sewage treatment, with especial reference to screens, filters, tanks, and the various structures and appurtenances used, and was impressed with the high efficiency of the screening plants, especially in Germany, and the great extent to which they have come into use there as an entire method of sewage treatment.

Although there are a number of fine screens in use in England, they generally form a part of a more extensive treatment, and their purpose is preparatory to other measures. The English rivers are small, and the population crowded together. Several large cities elbow each other on the same small stream which, from its source to its mouth, has to carry away the effluents of a series of disposal plants, located at short distances from each other.

In some places, as at Birmingham, the sewage-plant effluent is much greater in volume than that of the stream into which it is discharged. Such conditions require the highest grade of treatment, regardless of expense, and the most highly developed disposal plants in the world are the result. In Germany, the rivers are larger and usually have a swiftly flowing current, thus making disposal by dilution possible in many places after the removal of the grosser materials by screens. In England the settling tank, as a rule, is used as the essential reliance of every disposal plant, and some method of filtration is provided. In Germany the fine screen is making rapid progress as a complete method, a single type of screen having been introduced in more than fifty cities, not to mention many other types which have been used extensively. Even where tanks are used, such, for instance, as the Emscher tank, it is generally the purpose to remove, as quickly as possible, matters which will settle, and not provide long periods of septic action; such tanks, in fact, perform the service of screens to a

* Brooklyn, N. Y.

great extent, and do it with a high degree of efficiency. It might, perhaps, be successfully maintained that such tanks are water-screens, which possess the added advantage of being able to digest the screenings. German practice in sewage disposal, guided by scientific methods of study and management, aims at securing a satisfactory result, and does not justify the expenditure of public funds for obtaining a higher degree of sewage purification than this demands. Mr. Hammond.

There are only a few places in England where fine screens would give an effluent of sufficient purity to prevent nuisance, but, in Germany, cities like Hamburg, Dresden, Bremen, and many others, are sufficiently well served with screens alone; and, where tanks are used, as throughout the Emscher District, the period of sedimentation is short, and the effluent is comparable to screened sewage.

In Hamburg, through the kindness of Dr. Dunbar, Mr. Fort and the writer were given every opportunity to make observations on the operation of the screening plant and the condition of the Elbe River. As this screen is described by Mr. Allen, further comments are not necessary. It can scarcely be called a fine screen, in the sense that the Dresden screens are fine screens, but careful observations failed to reveal any floating material from the sewage in the river—and these observations, extending over several days in June, were made from Dr. Dunbar's launch and also from other vessels moving on the river.

The condition of the river, from the water-works intake for Altona, below Blankenese, several miles below Hamburg, to a point several miles above Hamburg, was examined and found to be good, with no visible evidence of pollution at any stage of the tide, Dr. Dunbar stating that it was the ordinary condition. From a boat on the surface it was so difficult to find the points at which the sewage enters the river that Dr. Dunbar was not sure that he had located them correctly until, after careful observation of landmarks, he was able to state that the boat was directly over the area affected by the discharge, and the writer would not have known it had it not been for this assurance. In as busy a harbor as that of Hamburg, in spite of all regulations to prevent pollution, it would not be possible to prevent the occasional dropping into the water from shipping of some floating substances derived from food under preparation, or cast away —the usual wastes incident to shipping and traffic. Such materials would be present if no sewage entered the river, and the writer marveled that so little matter of this nature was to be seen. The presence of sea-gulls has been cited by some observers to prove the presence of pollution, but, although the writer observed a few, he carefully noted that they did not seem to find any food, and he did not observe any of them near the outlets of the sewers or near the screening plant.

This plant was nearly perfect in operation; it was so clean, in fact,

Mr. Hammond. that the engineer who took Mr. Fort and the writer through it and explained it in detail, was accompanied by his daughter, a young lady, who aided him as interpreter.

The screens worked perfectly, operating and cleaning themselves automatically; and all handling was done by machinery. The screenings were kept under cover, from the plant to the covered scow in which they are removed, and no odor was noticeable.

While investigating the river, a number of side basins were visited, as they seemed to be places where floating matter might tend to collect and remain, but there was neither floating matter of consequence nor visible evidence of pollution. When one looks into the river, the water, as in nearly all very large rivers flowing through extensive alluvial plains, has a slight brownish-green color, but samples taken in glass show no trace of this color; a moderate quantity of green algæ growth shows on piles and bulkheads. It was stated by Dr. Dunbar and others that the oxygen content is always satisfactory. The volume of flow is large, the tidal range being between 6 and 7 ft., but the water is always fresh, and, after filtration, is used for the water supply of Hamburg and Altona.

Mr. Fort and the writer visited the Altona Water-Works, about 10 miles down stream from Hamburg, as the guests of an Altona official who had passed much of his life in the United States, and most of the day was spent in examining all parts of the works, then being enlarged and improved. The water is admitted, during high tide, from the river to a series of large settling tanks at the river level; from these, after precipitation, it is pumped to the top of the hill or bluff, 280 ft., and filtered. The filtration plant, originally installed and described,* many years ago, by the late James P. Kirkwood, Past-President, Am. Soc. C. E., has been reconstructed, the Jewel type of filters having been recently introduced. The large settling basins along the shore gave excellent opportunity for observing the average condition of the river which, 10 miles up stream, receives the sewage of approximately 1 200 000 people; and it must be said that it was satisfactory, there being no apparent signs of pollution.

After examining a considerable number of German sewage screening plants, the writer formed the opinion, which has been strengthened by reading Mr. Allen's paper, that a more extensive use of such plants in the United States, at small cost, would go very far toward the protection of our rivers from pollution, and under many existing conditions give all the necessary protection. Where the volume of water into which sewage is discharged is sufficient to take care of the finely divided impurities remaining after screening, no further treatment is required to prevent a nuisance, provided the screening plants are properly designed and carefully operated.

* "Report on the Filtration of River Waters", p. 120 (1869).

Mr. Hammond.

There remains the important problem of getting rid of the screenings. The writer was much impressed by this fact at Dresden, where the Riensch-Wurl screen, described so admirably by the author, seems to meet every requirement in sufficiently treating the entire flow of sewage, but where a pile of screenings, about 100 ft. long, 6 ft. wide and 6 ft. high, awaited removal, and gave forth no very pleasing odor.

The disposal of screenings is a subject of great importance, and the writer would be thankful for more extensive data on this aspect of the matter.

The removal from sewage of matters which settle, if this can be effected before septic action has intervened, seems to be indicated both by experimental experience and theory. Sewage, as it reaches the outlet of a well designed system, is ordinarily in a fresh state; if, under these circumstances, the organic matters capable of removal by a fine screen are taken away at once, their liquefaction in the sewage is prevented, and the ultimate demand for oxygen is greatly decreased. The sewage thus screened, while fresh, may have suffered some increase in colloidal matters in emulsified form, but this is of slight consequence in comparison with the benefit arising from the removal of such large masses of putrescible matter as these screens secure. Thus prepared, the screen-effluent may be given further treatment, as the situation demands, by percolating filters or contact beds, or, if the river flow is sufficient, it may be discharged through multiple outlets into the selected waterway; if it is desirable to protect shell-fish, it may be disinfected, and will require less disinfectant than a tank effluent which has had time to become septic.

It must be remembered: that the operation of a screen is positive, and must be accomplished with acting parts and a motive power; it will cease to operate if not carefully managed and kept in order; and it does not dispose of the screenings, but rather gives rise to a new but smaller problem than the one it is intended to solve. It solves a large problem, but creates the smaller one relative to the disposal of the screenings.

In comparison with this, a well designed Emscher tank is quite as efficient as a screen in the removal of matters which settle, and will take out as great a quantity, in parts per million, in from 20 to 30 min. retention, and do it in such a manner that the effluent will be in a better condition for subsequent treatment than that from the screen; and, furthermore, there need be no screenings to dispose of, as sufficient storage capacity can be provided in the digestion chamber to take all settleable and screenable matters and hold them without nuisance for 6 months at a time. Moreover, this tank does not require the high priced attention and care which must constantly be given to the screen and its operation, and when the digested materials are removed, they have been reduced to a small fraction of their

Mr. Hammond. original volume, and are incapable of causing a nuisance by bad odors. The Emscher tank is considered undoubtedly the greatest advance in sewage treatment in recent years. However, it is not without some difficulties, high cost of installation being the first; and frequently its operation is possible only if the sewage is pumped, which adds a positively operated unit to this method and increases the cost greatly; and although it will operate with a minimum of care, it must have that care regularly and constantly. Sludge drying beds must be supplied, or the sludge will have to be removed wet in portable tanks; but it is thought that this will not prove as great an expense as the removal of fresh screenings from a screen plant; the latter must be removed in a similar manner, in most cases, and are far greater in volume.

The selection of a screen plant or a tank plant for a given project must be determined by the conditions and the requirements of the case; and the sanitarian must look at the matter from a number of angles before he decides which to use.

It seems to the writer that the fine screen plant is indicated for extensive use in great seaboard cities, and in cities on large bodies of water, in which screening will prove sufficient for local conditions, and will give complete satisfaction. These screens will also be used where sewage farming in any of its forms is adopted, and in connection with sewage disposal plants for the purpose of removing scum-forming material from the sewage previous to tankage, and also for protecting sprinkling filter nozzles from clogging.

Mr. Riedel. J. C. Riedel,* M. Am. Soc. C. E. (by letter).—Mr. Allen's paper will unquestionably become a classic on this subject. The data presented will furnish the basis for comparison with experiments, now being made and hereafter to be undertaken, in those cities and towns where fine screens may be used advantageously.

The time is rapidly approaching, if it is not already here, when cities and towns bordering on tidal streams will be required to screen all their sewage which, in many localities, is turned, untreated and unscreened, directly into the streams. One matter of interest to the writer, which receives slight mention in the paper, is the loss of head through the screens. It is obvious that this will vary with screens of different types, but it is believed that, if any data on the loss of head in any screen are available, many engineers would feel grateful if they were presented. The writer will thank Mr. Allen if, in his closing discussion, he will comment on this subject and present any additional data on the loss of head that he may have available.

Mr. Pitkethly. David T. Pitkethly,* Assoc. M. Am. Soc. C. E. (by letter).—Mr. Allen is to be congratulated for the admirable way in which his paper

* Brooklyn, N. Y.

is presented, and for its completeness and detail. The paper is a most valuable one to the Engineering Profession. The data presented are in good shape for use and comparison, and the bibliography attached is very thorough. Mr. Pitkethly.

One feature of screening, on which the data are somewhat lacking, is the loss of head through screens of various types.

After the removal of suspended solids by screens, the most essential part of the problem is their final disposition and the prevention of a nuisance by their accumulation. This is a most important detail in the operation of a screening plant, and more data on this subject would be of value to the Profession.

Where other means of treatment are in use, which is usually the case in cities having the separate system of sewerage, the major portion of the storm flow is generally passed directly into the nearest body of water without treatment, and, as large quantities of suspended solids are carried by storm-water, it would seem that screening this flow would be the proper means of clarifying it and thus rendering it suitable for disposal in water without endangering the condition of the body of water into which it discharges.

In Boston, where rough screening is in operation, and where the course of the sewage, on discharge at ebb tide, can be traced for miles by the floating solids, a finer type of screen could very well be placed, instead of the bar screens now in use.

In many instances, fine screens could be placed where settling and Imhoff tanks necessitate pumping, and where thoroughness of treatment is not essential, thereby reducing the first cost of the plant.

WILLIAM L. D'OLIER,* ESQ.—This paper is an able presentation of the subject of the development and application of fine screens for the clarification of sewage. Mr. Allen's very complete data indicate thorough research and study of the subject, and an intimate knowledge of the installation and operation of screens, obtainable only by personal observation of the apparatus in service. He has contributed to our records in English a valuable addition on the subject of sewage treatment. Mr. D'Olier.

That fine-screening is rapidly becoming a recognized method of sewage treatment is shown by the consideration it has received by such eminent authorities as Rudolph Hering, George H. Benzenberg, Edwin A. Fisher, and the late Emil Kuichling, Members, Am. Soc. C. E., in connection with the problem at Rochester, N. Y. The International Joint Commission mentions screening as an acceptable method, and numerous sanitarians, engineers, commissions, and boards of health of various States are approving it as suitable. The Metropolitan Sewerage Commission of New York City and the New York Sewer

* Philadelphia, Pa.

Mr. D'Olier. Plan Commission both endorse screening treatment as acceptable and sufficient for Greater New York.

Fine-screening of sewage, with disinfection of the effluent, is endorsed as a sufficient treatment to afford proper protection to the shell-fish industry.

With just appreciation of the possibilities of dilution, and the self-purifying power of water, or the spontaneous purification of water, acknowledged by authorities, and with due consideration of the vastly increased digestive power of water for screened as compared with raw sewage, as shown in the Thames, England, in the Elbe, Germany, and in home waters, the treatment of sewage by fine screens will be deemed acceptable, and the effluent may be discharged safely into most of the waters of the United States. Numerous coast and river cities offer a very large field for the application of independent screening plants.

The speaker believes that Mr. Allen has not been sufficiently liberal in his statement that the field for fine-screening is distinctly limited. It must be admitted that this has been so, and has been due to a lack of appreciation of its possibilities, but, with the operation and results of tank-treatment plants, better known from actual experience throughout the United States, the Engineering Profession has realized the lack of entire success with, and certain disadvantages in, tank treatment, and, consequently, is more willing to consider other methods, hence the appreciation of the merits and possibilities of fine screens. Therefore, the field is broadening. Not only is the screen accepted as auxiliary apparatus, but, more particularly, as apparatus for independent treatment.

When it is considered that the foul matter in sewage is mostly solids, and that sewage treatment primarily consists of the removal of solids, the sooner they are removed the more complete is the removal. If they are removed from sewage while fresh, the pollution of the liquid mass is prevented; that is, the solids, particularly fæcal matter, are not retained to be dissolved and comminuted in time and with travel.

Absolutely fresh sewage may be filtered, producing an effluent clear and stable, thus indicating the importance of early treatment to prevent pollution.

The author's statement that, of the suspended solids removed by screens, two-thirds or more consist of organic matter, may be bettered when fine-screening is accomplished at the proper point and time.

Tanks operate on sedimentable solids; this is particularly true in present-day practice, with the discontinuance of chemical coagulants and precipitants. Therefore, tanks operate only on a percentage of the total solids in suspension, generally from 60 to 75% of the total solids

being sedimentable. Results obtained with settling-tank plants in the United States during the past year show a removal of approximately 50% of total solids in suspension. Mr. D'Olier.

Although the percentage of removal of solids has been lessened in tank treatment by the discontinuance of chemical treatment and the shortening of the detention period, it must be remembered that fine screens have been improved in design and construction, and that they act, not only as a screen, but as a filter, removing a percentage of non-sedimentable as well as floating solids.

Fine screens may be termed "classifiers," as they insure an effluent which varies only slightly in retained solids. A screen will take out all solids, sedimentable and non-sedimentable, larger than a certain predetermined size, and is not affected by any variation in flow or quantity of solids contained in the sewage. In any tank system the reverse is true; for instance, if a tank will remove 50% of the sedimentable solids, those remaining in the effluent are bound to vary in direct proportion to those in the influent, in addition to the non-sedimentable solids which the tank will not remove; this is also affected by any change in the detention period on account of the varying rate of flow. The condition of the effluent is an important feature.

Having determined the degree of treatment necessary, it should be maintained, regardless of any increase in the volume of solid matter. Screens will deliver a constantly uniform predetermined effluent. It is a difficult matter to ascertain what percentage of removal of non-sedimentable matter is effected by a screen, as the generally accepted method of reading tank results—the cone measuring glass—shows only sedimentable solids. This method has been generally accepted for reading all screen results, hence, in test readings, no credit is received for the removal of non-sedimentable solids.

Mr. Kuichling, who was a student of and advocate of fine screens, has stated that "properly designed and operated fine screens of one millimeter openings will remove as high a percentage of solids as would the best designed tanks with 1½ hours detention." To appreciate the truth of this statement, it must be borne in mind that the fine solids—those which have been comminuted to extreme fineness—are those which tanks will not remove successfully, and which cause the greatest trouble in the consumption of oxygen, thus affecting the sewage and the water into which it is discharged.

Further, in order to accept Mr. Kuichling's statement regarding the operation of fine screens, it must be fully determined what the general statements regarding the percentage of removal of solids by tanks really mean. Such general statements as "tanks will remove 90, or 80, or 70%" (at times followed by modifying clauses), may be quite misleading unless considered carefully and intelligently. Such a per-

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Mr. D'Olier. centage of removal of some classes of solids may be accomplished, but not of the total solids in suspension. Careful reading of general statements regarding the removal of solids will show that some factor must be used to determine the full percentage of removal of total solids, or, that the tests have been conducted inefficiently or incorrectly.

There is every reason for the author's suggestion that prescribed methods of tests and readings should be established.

The speaker has observed effluents from tank-treatment plants, which could have been improved materially by proper fine-screen treatment.

Plants for fine-screening may be more conveniently located and will require very much less area than those for tank treatment, effecting invariably large savings in the construction of long discharge sewers or interceptors, and of ground area.

With tank-treatment plants it is invariably necessary, or deemed prudent, to purchase extensive land tracts, representing large land costs, in addition to miles of connecting sewers or force mains necessary to reach remote sites.

In addition to the large field available for independent screening plants, a large and extensive field for fine screens is their use as primary or auxiliary devices in extensive treatment. It is generally considered good practice to place ½ or ¼-in. screens on lines leading to tanks, as protection to apparatus, piping, valves, etc., against mechanical obstruction. The advisability of using screens as an aid to tanks is apparent when due consideration is given to the non-digestive nature, or the tendency to resist tank digestion, of various solids which have been generally supposed to submit quickly to digestion. The introduction of such materials into the digestion chamber not only occupies space which has been provided at far greater expense than screens would require, but retards digestion in the entire chamber.

Exhaustive tests and studies of this subject, made during the last several years by Dr. Dunbar, Dr. W. Favre, Dipl. Ing. Spillner, and Dr. Guth, have furnished definite data and knowledge of the digestive and non-digestive organic substances, which, it might reasonably be supposed, could be properly deposited in tanks, but which, the results of the experiments would indicate, should be screened out, in aid of tank operation. In many American cities, appreciable quantities of leaves, stems, twigs and match sticks—all vegetable substances which will not readily submit to digestion—tend to burden tank operation and delay digestion. Further, hair and light materials, which tend to form scums, seriously retarding the proper operation of tanks, may all be removed by ⅛-in. screens.

The value of fine screens, following tank treatment, to protect sprinkling nozzles, is shown by their successful operation at Balti-

more, where the labor necessary to keep the nozzles clear is reduced materially when the screens are in operation. Mr. D'Olier.

An application of fine screens which has not yet been adopted, though suggested by several engineers, is that for raw sewage, with finer screens following chemical treatment.

At present, storm-water is discharged without treatment, but this matter has recently been receiving more consideration and endorsement; for such service, fine-screening is particularly well adapted. An important reason for treating storm-water is the fact that it is distributed over a far greater area than the general or dry-weather flow, and thus pollution carried by it goes beyond the limits or confines in which it may be feared or guarded against.

Progress in this matter in the United States has been prevented by the past unsuccessful operation of fine screens of American design, which have lacked features essential to success both in design and construction. Designs must be made with due consideration of the results desired. The removal of solids below the flow of the sewage, and the action or travel of the screen against the flow, produce comminution, and are due to incorrect principles. The important features and advantages of properly designed and constructed fine screens are: Low initial cost; low operating and maintenance costs; application to large or small installations; serviceability as fine or coarse screens; maximum effective screen area; operation under various flow levels; minimum loss of head; effective screen seal; slow speed; small space; availability for existing outfall sewers; noiseless and cleanly operation; screen chamber affording no recesses or corners for accumulation of solids; continuous, automatic, and instantaneous removal of solids, avoiding putrefaction; removal of heavy, true suspended, light, and floating matters; recovery of fats and grease; low velocity of travel of screenings through sewage; minimum disintegration of screenings; non-septic tendency in course of treatment; screenings of a consistency easily handled; and possibility of recovering grease and manurial values in screenings.

The term "screenings" should be accepted as denoting solids removed by fine screens, these being fresh solids containing manurial values. The term "sludge" should denote solids discharged from digestion tanks, with their values spent.

It is acknowledged that screenings possess considerably greater values than sludge, and are delivered in a spadeable form, with a water content of from 75 to 80%, as compared with sludge, in a liquid form, with a water content of 90% or more, necessitating a sludge bed or mechanical de-watering.

The volume of screenings *versus* the volume of sludge may be realized by reference to Fig. 33, which compares their relative volumes, and is based on the data in Table 25.

Mr. D'Olier. The daily production of screenings, and the resultant necessary disposition thereof, will have on the whole a great effect on the care and operation of treatment plants. A plant of any type will not operate successfully without intelligent and continuous care. Contrary ideas are responsible for the deplorable condition to-day of numerous sewage-treatment plants throughout America. The Engineering Profession should impress particularly the need of intelligent care and operation.

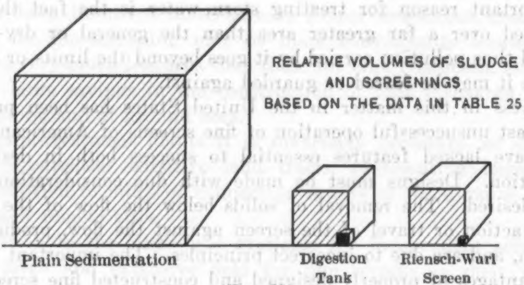

FIG. 33.

TABLE 25.—DATA RELATIVE TO REMOVAL OF SOLIDS FROM SEWAGE BY SEDIMENTATION, TANKS, AND SCREENS.

Method.	Percentage of total solids removed.	Percentage of moisture.	FROM 1 000 000 GALLONS OF SEWAGE. Cubic feet of dry solids.	Cubic feet of wet sludge.	Tons of wet sludge.
Plain sedimentation (2 hours)..	53	95	18.5	370 @ 63 lb.	11.6
Digestion tank (1 hour).........	50	90	17.5	175 " 64 "	5.6
Digestion tank..................	50% digestion.	90	8.8	88 " 64 "	2.8
Digestion tank..................	Dry on sludge bed.	60	8.8	22 " 68 "	0.75
Riensch-Wurl screen............	50	80	17.5	87½ " 66 "	2.9
Riensch-Wurl screen............	Burn in destructor (ash = 10% of weight of wet screenings).				0.29

In Russia, at certain treatment plants, legal requirements prohibit the discharge of solids greater than a certain diameter into public waters; the sewage is screened; the screenings are ground or disintegrated so that they will be of less than the specified size, and are redeposited in the discharged effluent. Such a method is utterly useless, and is on a par with some attempts to discharge undigested or digested sludge with the effluent from treatment plants. Under no conditions should such disposition be considered or permitted. Where such a practice is permitted, a resultant deplorable condition will soon manifest itself.

The method of removing solids from the screen is important; any hoeing or raking action will force them through the meshes or openings. Hydraulic methods are not effective, as they do not remove fibrous matter, and are objectionable on account of splashing and also the additional water content of the screenings. A continuous revolving brush, covered with a hood connected with air suction, is undoubtedly the most effective and sanitary. Mr. D'Olier.

In the design and mechanical construction of screens, due consideration has not, until recently, been given to the stresses and strains resulting from service under the hydraulic head frequently encountered under working conditions, with the result that invariably, with American installations, there have been serious difficulties, either leading to abandonment or necessitating reconstruction; these defects the Germans have remedied with screens of several types, so that the service is continuous and satisfactory.

The Riensch-Wurl screen, now in operation at Dresden, Bremen, Eberswalde, and numerous other places in Germany, as well as in Norway, Russia, and France, has, in the past five years, met and fulfilled all the requirements. Its recognition and adoption in the United States have led to the purchase of the invention and its manufacture in America.

In connection with the adaptation of the design of the screen for American service, important features suggested by experienced civil engineers have been incorporated, and have insured its satisfactory operation. It has the largest available screening area, permitting a reduced velocity in the flow; it operates in the flow of the sewage, gently, gradually, and continuously, raising the screened and settled solids to a position where they may drain and be swept from the screen plate by a correctly designed brush, in a manner which absolutely removes the solids without forcing them through the screen openings, as shown by Fig. 34. The design of the brush and its operation may be better understood by reference to Fig. 35.

The screen plate is smooth, with tapered openings, insuring free passage without obstruction, as shown by Fig. 36.

The screen acts as a skimmer, in removing a maximum percentage of grease, and, with sewages containing high percentages of this substance, will occasionally require freeing by steam, gasoline, naphtha, or other solvent. The bearings are above the flow, and are easily accessible for inspection and repair. All parts of the screen structure are readily brought above the sewage flow for care and attention. In the bascule design, the entire screen may be raised out of the flow of the sewage.

The power consumption is low, and also the operating speed of the screen, and may be varied and controlled automatically. The brush-

Mr. D'Olier. cleaning mechanism works in a fixed ratio to the screen speed. The loss of head is a minimum. The screen seal, as shown by Fig. 37, prevents leakage and insures fine screening of the entire flow. At Dresden, Germany, one man takes care of the entire station of four 26 ft. 3-in. screens. The size of the screen-house may be noted from Fig. 38. In design and construction, due consideration has been given to the varied and at times extreme thrust and strains caused by heavy loss of head.

Mr. Allen's compilation in Table 11 was based on data available from tests made under such varied conditions as to render the comparison of no real value. An efficiency of 90% for the band screen at Göttingen is questioned. An efficiency of 63% for the Jennings screen is the result of operation on stock-yard wastes, and therefore is not comparable with domestic sewage. An efficiency of 10% for the wing screen at Frankfort is attributable to the presence before the screen of an effective grit chamber and to the large openings in the screen, which is of the bar type. At Strassburg and Gleiwitz efficiencies of from 10 to 63% are explainable by the different types of installations. The 53.5% efficiency of the screen at Mainz is the result of experimental operation only. The efficiencies of the Weand screens at Reading and Brockton (42 to 71.3%) are due to the difference in design of the plants.

The efficiency of the Riensch-Wurl screens at Dresden, scheduled at 33.6%, is the result of one method of test. Numerous trials of these screens, in some fifty plants, show from 32 to 86%, averaging more than 50 per cent.

So many factors have such a serious effect on test readings as to render them of no practical value.

Tables 9 and 10 will be better understood with the following explanation: Table 9 shows a cleaning effect of 66.4%, as the average for a year, and Table 10 an average of 33.64 per cent. In comparing the two logs, attention is called to the fact that the screen resolves itself into a classifying apparatus. It will be noticed that the solids in the effluent in both cases were about the same, and that the influent varied about 100% in its content of solids. Furthermore, as the author states, the data in these two tables were recorded under differently prescribed methods of sampling, which has a great influence on the results obtained, by screens or any other method of sewage treatment.

The earlier practice at Dresden was to draw samples of sewage, both influent and effluent, in a pail, collect a quantity, and then draw a composite sample, and, with that make a cone glass reading, as is usually made for tanks. For a period of 3 months daily recorded tests varied from 33 to 86%, averaging 66+ per cent.

FIG. 34.—SECTION OF RIENSCH-WURL SCREEN, SHOWING SCREENINGS COMING UNDER THE ACTION OF THE BRUSHES.

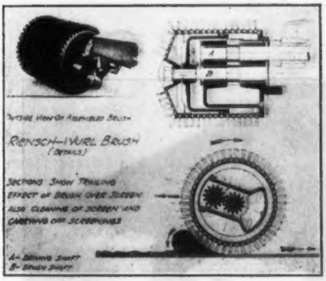

FIG. 35.—SECTIONS OF BRUSH, SHOWING ITS TRAILING EFFECT OVER SCREEN, ETC.

FIG. 36.—SCREENING PLATES OF RIENSCH-WURL SCREEN.

FIG. 37.—ADJUSTABLE SEAL OF RIENSCH-WURL SCREEN.

FIG. 38.—EXTERIOR OF RIENSCH-WURL SCREEN-HOUSE, DRESDEN, GERMANY.

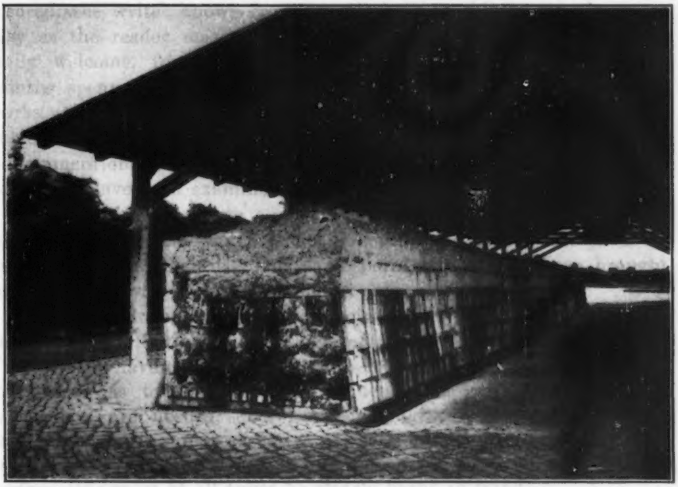

FIG. 39.—SALE SHED AND BINS FOR STORAGE OF SCREENINGS.

With the distinction between sludge and screenings, to which reference has already been made, their handling and disposal will be of interest. Tank sludge, with its 90% (or more) of moisture, requires re-handling in a de-watering stage, and is ultimately of no value, so that its disposal is in every sense an item of expense. Screenings containing 80% (or less) of moisture must be handled promptly. Disposal by incineration, as suggested by the author, is sanitary and effective, but the possibility of at least some return is being considered more and more. By-product processes are developing. Grease, ranging from 8% to greater quantities, may be recovered profitably and produce a further de-greased dry sludge by-product, valuable as a fertilizer. Mechanical de-watering of sludge, incineration, and by-product recovery from screenings are all established methods. Fig. 39 shows the screenings at Dresden stored and awaiting delivery to farmers. The city formerly disposed of all its raw screenings for $500 per year; with its by-product recovery plant, the returns have been increased to $2 000 per year. Mr. D'Olier.

JOHN H. GREGORY,* M. AM. SOC. C. E. (by letter).—The author has presented a paper which cannot fail to be of great interest to all who in any way have to do with the screening of sewage, and he is to be congratulated on having been able to secure so much detailed information on foreign works. That such information can be secured, the writer knows from experience, but this is not always as easy as the reader may imagine. To the writer, the paper is especially welcome, for it brings back pleasant recollections of many months spent on the Continent and in England examining similar works. It was his good fortune to be abroad in 1909, and again during the summer of 1914, and although he has not visited all the works mentioned by the author, he is familiar with many of them, and many have been examined twice. Mr. J. H. Gregory.

That fine screens have a definite field in sewage disposal cannot be gainsaid, but the writer is not so optimistic as the author as to the results accomplished. This same point has already been brought out by others who have discussed the paper.

The paper brings up so many points that a discussion of all is hardly feasible, but there are a few matters which may be mentioned. Fine screens have come to stay, but, before trying to draw too definite conclusions as to their efficiency, much more information and detailed data are needed. All sewerage engineers recognize the difficulty of securing representative samples of sewage as it arrives at or passes through works and, in judging of the work accomplished by screens, this must at all times be clearly borne in mind.

* New York City.

Mr. J. H. Gregory.

Many of the author's data are from European works, and in studying the results obtained it must be remembered that there is a great difference between most European sewages and those generally met with in the United States. In Europe, the sewages are, in general, very much stronger than those in America, perhaps, on an average, three times as strong, and, in the writer's opinion, it does not necessarily follow that as good results can be obtained in screening the weaker American sewages as seem to have been obtained with the stronger sewages abroad.

In comparing the efficiency of fine screens and settling tanks, the writer would place a higher relative efficiency on properly designed and operated settling tanks than the author appears to have done.

There is one feature in connection with the operation of screens which the writer feels should receive much more consideration than the author seems to have given it, that is, the handling and removal of the screenings. Screenings rapidly become offensive and, unless removed quickly, may become a source of great offense at the works. This has been forcibly brought home to the writer when examining some of the works cited by the author. The handling and removal of screenings from a screening plant is just as important as the handling and removal of sludge from settling tanks, and provision must be made for doing this promptly if offensive conditions are to be prevented. In many works which the writer has examined the question of the handling and removal of the screenings seems to have been of secondary consideration.

It has been suggested in one of the discussions that, in the treatment of packing-house wastes, by the use of fine screens in connection with settling tanks, a saving in the sludge storage capacity of the settling tanks and in the area of sludge drying beds could be secured, and that thereby a substantial saving in the cost of tanks and drying beds should result. Under some conditions this might be the case, but the saving may be more apparent than real. The saving in cost of tanks and drying beds, in part, at least, would be offset by the additional cost of the fine screens, and at first sight it would not appear reasonable to reduce the size of the settling chamber, as the removal of a portion of the suspended matter would not affect the volume of the sewage appreciably. In fact, the addition of the fine screen may increase instead of decrease the cost of the works.

Again, cost of construction is not the only factor to be considered. There is the cost of operation of the fine screen and the removal of the screenings which must be taken into account, to say nothing of the fact that screenings removed from packing-house wastes are perhaps of the most offensive kind, and if removed from the sewage, the likelihood of nuisance would be greatly increased.

Mr. J. H. Gregory.

As stated previously, screenings quickly become extremely offensive and, from the writer's observations, the surest way to prevent nuisance from such materials is to keep them away from the atmosphere, and the simplest way to do this would appear to be to keep them in the sewage and handle them in the settling tanks. Minimum cost is not the only desideratum in building a sewage disposal works, absence from offense is generally equally important.

The use of the fine screen in connection with settling tanks to prevent the formation of scum is another matter, and it may be that here the fine screen could be used to advantage, but it seems to the writer that the data are yet too meager to enable one to draw definite conclusions on this point.

The writer feels personally indebted to the author for having presented such an interesting paper and for having brought together information and data which have been so scattered.

Mr. Soper.

GEORGE A. SOPER,* M. AM. SOC. C. E. (by letter).—Credit is due the author of this paper for calling attention to a form of apparatus for the treatment of sewage concerning which, thus far, little notice has been taken in the United States or England. To many persons connected only with the practical side of sewage works, it will be news that such apparatus as these highly developed screens exist; and yet there is nothing especially new or newly discovered about them. They have been seen for years by all American engineers who have made tours of inspection of European sewerage works.

There has been little inclination to use German screens in the United States, partly because it was not certain that they could accomplish satisfactory results under the conditions which obtain here, and partly on account of the fact that the apparatus was mechanical, proprietary, and not capable of convenient, prompt, and economical repair.

Until recently, no German screen has been pushed commercially in the United States, and few have thought it worth while to translate into English more than fragmentary descriptions of it. Mr. Allen has performed a useful service in bringing together a great deal of information which cannot fail to arouse American interest in a very important type of sewage apparatus and lead to discussions, both in and out of this Society, which will broaden the horizon and add to the resources of sewerage engineers.

It is to be hoped that this paper will not lead to a too enthusiastic view of the functions of screens. The writer has visited many German works, is familiar with the best installations, and is a strong advocate of screens in their proper places; but he thinks that American engineers should regard them with a great deal of conservatism until they

* New York City.

Mr. Soper. have been well tried. The conditions under which they operate in Europe are quite different from those under which they would be compelled to work here. It is unwise to infer that the foreign results which are reported could be duplicated. It is possible that the German screens can do better here than anywhere; but it is far more likely that they will find their best development in adaptations and improvements which American investigators and American ingenuity can devise. These remarks refer particularly to the more complicated types of screens; a few of the simpler forms seem suitable for use in America with but little change.

So far as the writer sees, screens are sufficiently different from other types of sewage apparatus, and have such individual functions, that they are warranted in being placed in a class by themselves. If this view is correct, it is unfair, both to them and to settling basins, to regard the two as though they were competitors; and yet they are competitors, in the sense of being able to improve sewage by the removal of a part of the solid matter; and comparison for the purpose of bringing out their differences is plainly desirable.

Screens are mechanical contrivances which, for proper operation and repair, require constant and skilled attention. If one of their many moving parts gets out of order, the whole screen may become useless. A very small accident may put one out of operation for a long time. In Germany the labor which is employed in connection with sewage disposal works is better and cheaper than that customarily seen in America, and as attendance plays such an important part in the operation of screens, in comparison with settling basins, the latter have an advantage in this country. Settling basins are suitable for installations of large and of small size, but screens can be used only where the plant is sufficiently large to permit of the expense of proper attendance. The material taken out of the sewage by screens must be removed from the plant every day, whereas that from settling basins can remain for long periods. In the case of some screens, the cost of depreciation and repair must be large, but, unfortunately, there are few figures from unprejudiced sources which can be quoted in this particular. It is trifling in comparison with settling basins.

The results of operation, under the best circumstances, are quite different with screens and settling basins, and although some of the tests which have been reported seem to indicate that they may both do about the same work in the removal of organic matter in sewage, a moment's thought will show that they produce different effects on the sewage. Screens remove particles according to size, settling basins according to weight. Neither approaches the ideal, which is removal according to composition. Screens, however, have some advantage, inasmuch as a large part of the heavy solid matter which goes to the credit of settling basins is not putrescible. On the other hand,

much of the material which floats will not decompose rapidly enough to make trouble if left in the sewage, so that the difference between screens and settling basins on this account is not great. Screens have the disadvantage of breaking up some of the very material which it is most desirable to remove, so that it has been claimed that there was more organic matter in solution in the effluent than in the applied sewage in some plants. There is no such action in settling basins, but the latter have a compensating disadvantage in the fact that they keep the sewage on hand several hours longer, thus favoring decomposition. Mr. Soper.

So far as offensiveness in operation is concerned, the advantage appears to be in favor of the screen when scrupulous care can be taken with it. When operated without regard to the production of odors, screens are undoubtedly among the most offensive kinds of apparatus used in sewage treatment. The liquor contained in the screenings is peculiarly offensive.

American practice certainly should not tolerate the handling of screenings as is done at most of the German works. They should be burnt as soon as possible, unless they are to be utilized by conversion into fertilizer at a plant which is operated under skilled direction. In this event they should be stored temporarily and transported from the screen works in closed and tight containers. It is a mistake to suppose that screenings would have a value in America commensurate with the nuisance which they would create. American farmers do not need such material, and when they get it they do not know how to use it.

Chemical analyses should mislead no one to form the idea that the nitrogen is so valuable that it will well repay a farmer to make use of it. He can get the nitrogen which he needs for much less money in far more convenient form by buying it. The writer is of the opinion that the recovery of a part of the manurial ingredients of sewage is both practicable and desirable, but only in those cases where the works are sufficiently large, well managed, and so situated, as to permit the work of recovery to be carried on to advantage.

Screens have an advantage over settling basins in the fact that they can properly be installed in built-up sections of cities, where it would be inadmissible to place settling basins. This is possible by reason of the smaller space which they take up, the less chance of nuisance, and the less objection which is likely to be made to them by property holders in the vicinity. Well-built settling basins, operating on modern principles, do not produce a nuisance, according to the writer's observation, when placed where there is perfect freedom of ventilation, but there is grave doubt as to their inoffensiveness otherwise. It has sometimes been proposed to place settling basins beneath the streets of cities, and in closely covered buildings in the

Mr. Soper. midst of thickly occupied business and residence districts. There seem to be too many risks, such as those of explosion, of offensiveness, and of popular objection, to make this proceeding wise. On the other hand, it has been shown to be practicable to build screens in compactly settled districts. This is a particularly suitable field for them, if they can be managed in such a way as to avoid nuisance.

Screening is useful according to the location of the screens. The fresher the sewage, the more effective will be the process. To place fine screens at the end of a long sewer, as is done in some of the German plants, is to limit the quantity of material which they can remove, or to make it necessary to provide screens of very great fineness. More than this, the very kind of material which it is most desirable to remove from the sewage becomes in large part broken up by the friction and submersion in long conduits, and resolved into the troublesome colloid state in which it escapes screens. The Germans are well aware of this, and have placed relatively coarse screens in the built-up section of Hamburg and fine screens at the end of the long outfall at Dresden. It is not probable that these plants would operate at all well if their locations were reversed.

The two most promising uses of screens, in the writer's opinion, are in preparing sewage for direct discharge, under suitable conditions, into natural bodies of water which have sufficient capacity to digest it, and in taking from sewage those solids which can be removed in relatively large quantity, when fresh, and which would be broken up and add materially to the cost of disposal at the end of a more or less distant main sewer.

It should not be supposed that screens of any degree of fineness clarify sewage, in the sense of making it clear or reducing the greasy or discoloring effects which are produced when it is discharged into streams or other natural bodies of water. It can lessen the offensive appearance only to the extent of taking out those particles which are separately recognizable as of sewage origin.

The fact is, clarification, like many other terms used in sewage treatment, is a misnomer. This is fairly well understood, and on this ground, the author is justified in using it, although its continued use is certain to lead to a wrong impression in many quarters.

Similarly, the term "fine screens" has no definite meaning. The author has undertaken to limit the application of the term "fine" to screens which have a clear opening of 0.6 in., this limit being adopted, apparently, in order to include the well-known Hamburg works. In this distinction, he will not always be followed. There is nothing about the size of the opening which should distinguish one kind of screen from another. The manner of cleaning has more to do with it. As all the screens which have been described in the paper are cleaned with little or no hand labor, the term "self-cleansing screens",

or "automatic screens", would be, at least, as appropriate; but why "fine", or "self-cleansing", or "automatic", unless for commercial purposes? In the present state of the art, the simple term "screen" should be sufficiently descriptive. Mr. Soper.

In considering the use of screens, it should be remembered that the screen itself is but a part of the plant which is needed. Screens of coarser size are required in order to protect the finer apparatus against injury and the impairment of its proper function. These must be cleaned, a process which is usually done by hand, and in a simple and crude manner. Grit chambers are needed to perform a like office with respect to the heavy particles, which are recoverable and may do mechanical injury to the screens. Power must be provided to operate the screens, or, at least, their cleansing devices. Storage bins must be supplied for the reception of the screenings, and other provision must be made for their transportation or incineration. Shelter must be given to the plant.

The efficiency should be taken as the action of the whole plant, if the efficiency of screening is to be understood by others than experts.

It is unfortunate that the ways of sampling, testing, and expressing the efficiency are so diverse and unsatisfactory that it is difficult for any one to form a clear conception of what screens are capable of doing. It is a serious question how to test screens and settling basins, and by what criteria to judge them. None of the customary methods seems adequate. The composition of sewage is so various, and its physical, chemical, and biological constituents are so changeable and changing, that it has thus far been impossible to find satisfactory means of determining it. Terms such as "solid matter", "suspended matter", "settling solids", and "suspended organic matter" have very little significance, apart from the behavior of the sewage in some particular respect, in some particular manner, or in some particular case. Laboratory expressions are often misleading. Some of the most dangerous-sounding terms are entirely robbed of their supposed significance when translated into simple language. For example, "nitrogenous organic matter in suspension", may be hair, than which there are few things more incapable of creating foul odors in sewage or of calling for expensive purification works.

The author has performed a difficult and painstaking task in getting together his admirable descriptions of nearly all kinds of screens, whether coarse or fine, and his data are of unusual interest; but the lasting value of his work will not lie in the tables of efficiency. Such data need to be determined by Americans, and for American conditions. It is earnestly to be hoped that those who have charge of sewage experiment stations will see their way clear to install screens of various types and operate them with some not too exceptional sewage for a sufficient length of time for the relative merits of the

Mr. Soper. different forms to become better known. For average sewage, the three forms which have impressed the writer as the most likely to meet practical requirements are those used at Hamburg, Dresden, and Frankfort.

It is acknowledged everywhere that American engineers have done an immense amount of good work in testing out sewage purification processes experimentally, on both a large and small scale, and the reports of some of their investigations are among the most valuable contributions in the literature. Intermittent filters, septic tanks, settling basins, contact beds, and sprinkling filters—all in great variety as to proportions, materials of construction, and rates of flow—have been brought out, have held attention as the leading process of the day, and have been finally placed in the position where they belong. Screening is the last call for notice, and it is to be hoped and expected that it will receive the attention and investigation to which its merits entitle it.

Mr. Allen. KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—The writer wishes to express his gratification at the interest shown in the subject of his paper as indicated by the number and character of the discussions brought out. In particular, these have emphasized: The necessity of standardizing methods of analysis, and the desirability of screening as near the source of pollution as possible.

The views expressed as to the efficiency of fine screening are quite diverse, and are particularly valuable at this time in showing the need of a standardization of procedure in the sampling and testing of sewage and screenings. The Laboratory Section of the American Public Health Association is working along these lines, and within a few years it may be possible to obtain results of screen operation that are safely comparable. The nine efficiencies recorded in Table 11 appear to be discordant. They were obtained from different sewages by different screens operated under different management, and the technique in determining the results was probably different in them all. For this reason the writer added the foot-note stating that "More complete data should be secured, in order to furnish a reliable comparison between the efficiencies of different screens." This is in accord with a statement of Mr. Pearse to the same effect.

Since reading the discussions, the writer is more than ever impressed with the futility of placing reliance on efficiencies as ordinarily published. Mr. Greeley, for instance, states that "The available data indicate that a removal of 15% has seldom been reached in practice." Messrs. Hering, Kershaw, Fuller, and Pearse believe 30% to be, in general, too high a figure; Mr. Stevenson mentions an efficiency of $\frac{200 - 133}{200} = 33\frac{1}{2}\%$, obtained experimentally with a 32-mesh screen at

* New York City.

Philadelphia. At Brockton from 50 to 70% is claimed, and Mr. D'Olier states that "Numerous trials of the [Riensch-Wurl] screens, in some fifty plants, show from 32 to 86%, averaging more than 50 per cent." These views are so conflicting that it appears more evident than before that we have much to learn with reference to the true and relative efficiencies of different screens. **Mr. Allen.**

The subject has been mentioned in so many of the discussions, however, that the writer thinks it desirable to present the following additional information, somewhat in detail, regarding the examples given in Table 11, in order to judge of the relative weight that should be attached to each.

Göttingen, 90 (?) per cent.—This figure was deduced by Mr. Kuichling* from data given by Frühling,† but doubt was expressed by the former as to its reliability, as indicated in Table 11. This result, therefore, may be eliminated from further consideration as probably too high.

Chicago Stock Yards, 63 per cent.—This efficiency was furnished the writer by Mr. C. A. Jennings, in a letter dated January 2d, 1914, as that of his screen operating on Stock Yards effluent. He says: "With an influent running about 700 parts per million of suspended matter, the screen removed about 63% * * * 79% moisture. * * * The above results are the averages of several tests made on the screen." There is no indication that this was a peak load, as assumed by Mr. Pearse. Nevertheless, it was obviously not a normal sewage—a fact which should be borne in mind in considering the subject. Much additional information regarding the operation of this screen may be found in the Report on Industrial Wastes from the Stock Yards and Packingtown in Chicago, by George M. Wisner, M. Am. Soc. C. E., Chief Engineer, and Langdon Pearse, M. Am. Soc. C. E., Division Engineer, dated October 15th, 1914, from which the following notes are taken.

The screen consists of 38 12 by 48-in. panels covered with 40-mesh Monel metal fabric. The net area was 112 sq. ft. With a speed of 125 ft. per min., this passed from 2.1 to 2.7 gal. of sewage containing 340 parts per million of suspended matter per square foot of clean screen. The percentage removed, or "computed reduction" was as follows:

October	23d	43%	November	5th	49%
"	24th	32%	"	19th	28%
November	4th	22%			——
			Average		33%

This was equivalent to 6 740 lb. of screenings, 83% moisture, or 1 150 lb. of dry matter per million gallons of sewage.

* "Modern Treatment of Sewage," *Proceedings*, New Jersey Sanitary Assoc., 1908.

† "Entwässerung der Städte," Leipzig, 1910, p. 522.

Mr. Allen. *Frankfort, 10 per cent.*—This is the figure given by Uhlfelder and Tillmans in "Die Frankfurter Kläranlage." Further information regarding the operation of the Frankfort screens has been received from Stadtbaurat Franze in a letter dated December 30th, 1914. Experiments made in 1908 gave the results shown in Table 26.

TABLE 26.—REMOVAL OF SUSPENDED SOLIDS BY GRIT CHAMBER AND SCREENS, IN MILLIGRAMMES PER LITER = PARTS PER MILLION.

	GRIT CHAMBER.		SCREENS.		TOTAL.	
	Total.	Organic.	Total.	Organic.	Total.	Organic.
Day sewage.........	65	39	25	24	93	63
Night sewage.......	61	32	5	5	66	37
24-hour sewage.....	64	37	22	19	86	56

"The detritus from the grit chamber is in equal amount organic and mineral, while that from the screens is almost entirely organic. That from the grit chamber is about one-fourth to one-fifth of the entire suspended matter in the raw sewage. The absolute quantities resulting from the foregoing experiments were as follows: [See Table 27.]

TABLE 27.—MATERIAL REMOVED BY GRIT CHAMBER AND SCREENS PER DAY, IN KILOGRAMMES.

	GRIT CHAMBER.		SCREENS.		TOTAL.	
	Total.	Organic.	Total.	Organic.	Total.	Organic.
Day sewage.........	3 252	1 614	1 317	1 138	4 569	2 752
Night sewage.......	557	292	55	48	612	340
24-hour sewage.....	3 809	1 906	1 372	1 186	5 181	3 092

"As the entire quantity of suspended matter in the sewage at the time of the experiments produced 23 549 kg. of dried substance, we see that the 5 181 kg. recovered by both the grit dredge and screen indicates that from one-fourth to one-fifth of the entire quantity of suspended matter has been removed by these preliminary operations.

"This detritus contains about 80% of moisture, the dried substance being 20 per cent. Considering this quantity of moisture, we have from 5 181 kg. of dried substance, a daily quantity of 25 905 kg. or 25 905 cu. m. (of specific gravity = 1.00) of screenings; or an annual quantity of 365 × 25.905 = 9 528 cu. m.

"Since the installation of the plant in 1905 the following quantities per annum have been removed: [See Table 28.]

TABLE 28.—MATERIAL REMOVED BY GRIT CHAMBER AND SCREENS ANNUALLY, IN CUBIC METERS. Mr. Allen.

Year.	Grit.	Screenings.	Total.
1905..........	3 604	3 610	7 214
1906..........	3 208	4 095	7 303
1907..........	4 365	3 720	8 085
1908..........	3 582	3 938	7 520
1909..........	3 859	4 407	8 266
1910..........	4 320	4 360	8 680
1911..........	3 745	4 562	8 307
1912..........	3 608	5 281	8 889
1913..........	3 062	4 774	7 836
Total..........	33 353	38 747	72 100

"In 1913 the total volume of sewage was 31 700 000 cu. m. We therefore have the following:

	Liters per cubic meter of sewage.	Cubic yards per million gallons of sewage.
"1. Detritus from Grit Chamber.....	0.15	0.743
2. Screenings	0.10	0.495
3. Total	0.25	1.238

"The percentage of suspended matter removed by these cleansing operations * * * can be seen in the following table." [See Table 29.]

TABLE 29.—PERCENTAGE OF SUSPENDED MATTER REMOVED.

	Suspended Matter.		Percentage.	
	Total, in milligrammes per liter = parts per million.	Organic, in milligrammes per liter = parts per million.	Total.	Organic.
Raw sewage..........	483	287	100	106
a. Grit..............	65	39		
b. Screenings	28	24		
Total *a* and *b*......	93	63	19.3	21.9

From the foregoing it is seen that the quantity removed per million gallons in 1913 was:

		Percentage of suspended matter. Total.	Organic.
Grit	540 lb.	13.5	13.6
Screenings	234 "	5.8	8.3
Total	774 lb.	19.3	21.9

Mr. Allen. These percentages are even lower than those given by Uhlfelder and Tillmans, but it must be remembered that the spacing in these screens is 10 mm. or 0.4 in., which is much coarser than any of the others (except Göttingen) for which efficiencies are given.

Table 28 shows that the screenings removed per day in 1913 averaged 13 cu. m. or 17 cu. yd. The figure originally given in Table 11 was evidently incorrect, and has been revised in accordance with this later information. The figure given by Mr. Greeley, 20 to 26 cu. yd., should also be corrected. According to Elsner* the screenings amount to 0.643 cu. yd. per million gallons, or 0.056 cu. yd. per 1 000 inhabitants daily.

With reference to the Frankfort sewage, it is, as mentioned by Mr. Greeley, quite fresh when received at the works. When seen by the writer it was a dark grayish brown, and the screenings consisted largely of feces and paper. The effluent, after passing through sedimentation tanks, flows to the River Main, the width of which is less than 500 ft., where it is diluted by 130 volumes of river water under the most unfavorable conditions of flow.

Strassburg, 10 to 12 per cent.—This percentage is stated as that of the material retained by the screen by Stadtbaurat Strohl in a letter dated May 9th, 1914.

Gleiwitz, 63 per cent.—This (62.9%) is obtained from the data in Table 4, furnished by Magistrat Muler. As stated in the table, the sewage contains "coarse material" and may, therefore, be fresh, explaining the high efficiency, or the latter may be due to the method of sampling.

Mainz, 53.5 per cent.—This is stated by Stadtbauinspektor Knauff† to be the efficiency obtained in a series of tests made by the city authorities, covering a period of several weeks. The sewage is not strong, containing "scarcely 1 ton per million gallons" or about 270 parts per million of suspended matter, but the percentage of removal varied from 46 to 61. These figures, being from an official competitive test, are believed to be entirely reliable, but, for the same reason, they are undoubtedly higher than would have been obtained in ordinary operation.

Reading, 42 per cent.—This figure is from an estimate by Mr. Kuichling,‡ showing a removal of 90 parts per million out of 215 parts per million of suspended matter, as follows:

"On a visit to the plant last year, the writer was informed that the weight of the screenings, after partial drying in a centrifugal separator, was 1 500 lb. per million gallons. This would make the condensed mass weigh 80 lb. per cu. ft., of which probably 50% is

* "Sewage Sludge," p. 17.

† *Wasser und Gas*, 1911–12, No. 8, p. 183, and *Engineering News*, April 26th, 1913, p. 470.

‡ Given in "Notes on Sewage Disposal," Rochester, March, 1910, p. 15.

moisture. If these figures are correct, the screen actually removed 750 lb. of fully dried matter per million gallons of sewage, or 90 parts per million by weight out of a total of 215 parts per million of suspended matter contained in the sewage on the average." Mr. Allen.

Mr. Fuller's statement that "on a basis of daily averages, this removal rarely if ever exceeded 30 parts per million" is quite at variance with the foregoing, but is probably based on a more recent and intimate knowledge of the operation of this plant, and it is, therefore, probable that the 42% as stated should be modified. The reason for this inconsistency with the data in "Sewage Sludge"* is that they were obtained from different sources.

Brockton, 71.3 per cent.—Regarding this screen, Mr. Charles R. Felton, then City Engineer, wrote on November 15th, 1912:

"The screen has been in continuous operation now more than a year, with practically no stops—not over an aggregate of two or three days in that time except the hour or two that we stop every day. The screening [mesh?] has not been replaced in that time. The time for cleaning the mesh occupies perhaps 15 or 20 minutes every morning. The detritus removed, in our case, amounts to from five to six thousand pounds per day after having been passed through a centrifugal dryer, from about 2 000 000 gal. of sewage. Our sewage, as you know, is very strong, on account of the water consumption, namely, about 40 gal. per capita.

"This five or six thousand pounds of screenings is equal to about four cu. yd. in that form. In our particular case it is necessary to raise the sewage into the screen, which is done by a 12½ h.p. oil engine which also revolves the screen. It is necessary to have one man constantly at the plant during the 24 hours, so that in working 8-hour shifts three men are necessary in the entire day."

The figures on which the efficiency were based were referred to X. H. Goodnough, M. Am. Soc. C. E., Chief Engineer of the Massachusetts State Board of Health, who states, April 9th, 1914:

"* * * the results as stated in our report for 1912, and as copied in your letter, are correct.

"There is danger of great error in conclusion about this screen unless the proper system of sampling is carefully carried out. If the samples are taken mostly in the day time they will represent a night flow of sewage and consequently a much weaker sewage than if the samples are taken throughout the 24 hours. We have some reason for thinking that the sampling was done with less care in 1913 than in the previous year."

Mr. Goodnough writes again, February 4th, 1915:

"The figures given in our report† showing screening results at Brockton in 1912 were made up by taking samples generally once each

* Pointed out by Mr. Pearse. Using 128 parts per million of supended solids, as given in "Sewage Sludge," would give $\frac{26}{128 + 26} = 17$ per cent.

† Massachusetts State Board of Health, 1912.

Mr. Allen. month, though there were, I believe, a number of extra samples taken in 1912. These samples were taken by employees of the Brockton Sewer Department at the pumping station, and each sample is made up of twenty to twenty-four equal portions collected approximately hourly throughout the twenty-four hours.

"The sewage strikes the screen with a good deal of force and, in consequence, the mesh after use for a time became broken in places, and this no doubt accounts for the poorer results subsequently obtained. Furthermore, the screen at Brockton is not so arranged that the entire screen is used, most of the screening being done by the first section.

"There was a change in management in 1913 which may have had something to do with the falling off in efficiency. There was again a change of management in 1914, and I send herewith the figures for 1913 and 1914, obtained in the same way as in 1913 [all in parts per million].

	Suspended Solids.			Albuminoid Ammonia.			Oxygen Consumed.		
	Raw.	Screened.	Removed.	Raw.	Screened.	Removed.	Raw.	Screened.	Removed.
1913	500.5	223.0	55%	16.4	12.4	24%	137.1	122.8	10%
1914	642.4	250.7	61%	21.6	12.5	42%	160.6	125.0	22%

"The authorities at Brockton took samples for awhile to check our own, but finding the results much the same, concluded that they would take our results and not take the trouble to carry on determinations to check them. So far as they did make observations, they appear to have checked ours very closely * * *.

"It is important to note in connection with the efficiency of screening in Brockton that Brockton sewage is one of the strongest in the State. * * * the amount of total solids * * * is exceeded only at Hudson and Norwood and * * * it exceeds somewhat the amount of solids in the sewage at Worcester. The amount of albuminoid ammonia also is much higher than the average. * * * In considering the results for 1914, it is necessary to take into account the fact that the year 1914 was a very dry one after May 13th, and you will note that the amount of albuminoid ammonia in the raw sewage examined in that year was higher than in any other, except the sewage of Hudson, while the total solids were higher than in any other sewage examined in that year.

"We have not had experience enough with screening to know its exact effect with different sewages. It is possible that the percentage of organic matter removed is higher with a very strong sewage like that at Brockton than would be the case with a much weaker sewage such as is found in some of the other cities of the State."

Further information regarding the Brockton screen is contained in a letter from Mr. B. R. Chapman, City Engineer, dated February 6th, 1914. He says:

"* * * our Weand screen has been in continual operation, nearly every day (since 1912), which gives a very limited time for proper repairs; the frame of the screen is now in bad shape and some of

the sewage does not pass through it at all, so that the reduction of suspended matter may be estimated at 50%. Mr. Allen.

* * * * *

"The cost of operation for 1914 was approximately $8 500, which includes attendance, oil for oil engine, water for cleaning screen, repairs, miscellaneous supplies, and also the carting of screenings to boilers to be burned.

"The capacity of the screen is about 3 000 000 gallons daily. This screen, I believe, has several mechanical defects and might easily be improved upon * * *."

A Weand screen operating on Center Avenue sewage (Packingtown), Chicago, was 4 ft. 8 in. long and 2 ft. 4 in. in diameter, with 29.3 sq. ft. of net area, covered with 30-mesh fabric. This was operated at a speed of 7 rev. per min., and received from 117 000 to 235 000 gal. per day, or from 4 000 to 8 000 gal. per sq. ft. of exposed screen. This is equivalent to about 0.4 to 0.8 gal. per sq. ft. of clean screen exposed. In twenty-seven experiments the removal of suspended solids amounted to 32% "computed" or 17% "actual" on day sewage. Continuing operation into the evening reduced the former to 12 per cent. There were removed by this screen 500 lb. of dry matter or 3 cu. yd. of sludge, 90% water, per million gallons.*

Dresden, 33.6 per cent.—This efficiency is by volume; the efficiency by weight is 30.95 per cent. Full details are given in Table 10, furnished by Baurat Fleck. This is the average result of a complete year of operation, samples being taken every fourth day at 4-hour intervals above and below the screen. Considering the care with which this plant is managed and the length of the test, the writer attaches more weight to the results obtained here than to those obtained elsewhere. The figures are corroborated by the results of the test of a Riensch-Wurl screen at Mainz (see page 935) where the efficiency, with a sewage of moderate strength, varied from 36 to 42 per cent. As to the distance from the source to the outlet, the situation in Dresden is probably not very different from that in the average American city. This may be seen by reference to Fig. 43. From Table 15, by Mr. Kershaw, there result the following seasonal efficiencies at this plant: spring, 18.7%; summer, 17.3%; autumn, 27.6%; winter, 48.3%; showing, in a rough way, the effect of temperature on the condition of the sewage.

The sewage when noted was a dark greenish-gray color. The screenings removed by the coarse screen were mostly composed of paper, rags, and sticks. Mr. Greeley's figures, 13 to 16 cu. yd. of wet screenings per day, do not agree with the volume reported by Stadtbaurat Fleck in a letter of April 9th, 1914, which was 19.7 cu. m. = 25.6 cu. yd.

* Report on Industrial Wastes from Stock Yards and Packingtown, 1914.

Mr. Allen.

Referring to Table 24, the quantity, 0.97, in Column 3, is in tons, and, adopting Mr. Greeley's assumption of 60 lb. per cu. ft., should be 1.20 cu. yd. This gives 12.4% removal instead of 10.1 as stated, if the parts per million of suspended matter are taken at 300. The authority for these assumed figures for weight per cubic foot and parts per million is not stated.

The writer believes that he is justified in giving preference to the official results of the year's operation stated in Table 10, as follows: 33.64% efficiency on the basis of volume or 30.95% on the basis of weight, rather than to a computation in which two of the factors appear to be assumptions.

Leaving the 90% of Göttingen out as unreliable, the 10% of Frankfort as inapplicable, on account of the coarseness of the screen, and, omitting the 63% of the Chicago Stock Yards, because of the abnormal character of the sewage, there remain six examples, in five of which, the efficiency as stated exceeds 30%, reaching 71.3% in the case of Brockton.

After making due allowance for errors of sampling and analysis, and considering that the efficiencies given are not selected, but a complete list of all the writer could find on screens of these types, he cannot agree with Mr. Greeley that "the available data indicate that a removal of 15% has seldom been reached in practice." It is certainly reached in Europe (for example, 66% at Wiesbaden*) and at Reading and Brockton, although, so far, experience in America is very limited.

The writer, therefore, believes that his assumption of a removal of 30% was not unreasonable for screens of the types described, having openings not more than 0.10 in. in size, under efficient operation. He believes, however, the 42% credited to the Weand screen at Reading is probably too high, and that data concerning efficiencies in general are far too meager to make any prediction with confidence. Higher values would naturally be looked for in Europe, with the stronger sewages prevalent there, but, as to length of carriage, conditions are probably not very different between European and American cities. Figs. 40 to 43† show the main lines of collection in London, Hamburg, Frankfort, and Dresden, from which can be obtained a good idea of the distance traveled.

This point, emphasized by Mr. Hering and mentioned also by Messrs. Fuller, Franze, Schaefer, Soper, and Kershaw, is believed to be very important. It is evident to any one who has observed the effluent from long outfalls or those from septic tanks. At Paris the 170 000 000 gal. per day received at Clichy is quite free from feces, the principal materials removed there being some 2 cu. m. (2.6 cu. yd.) of corks

* Weyl's "Handbook of Hygiene," Vol. II, p. 151.

† From Final Report, Metropolitan Sewerage Commission of New York, 1914.

Mr. Allen.

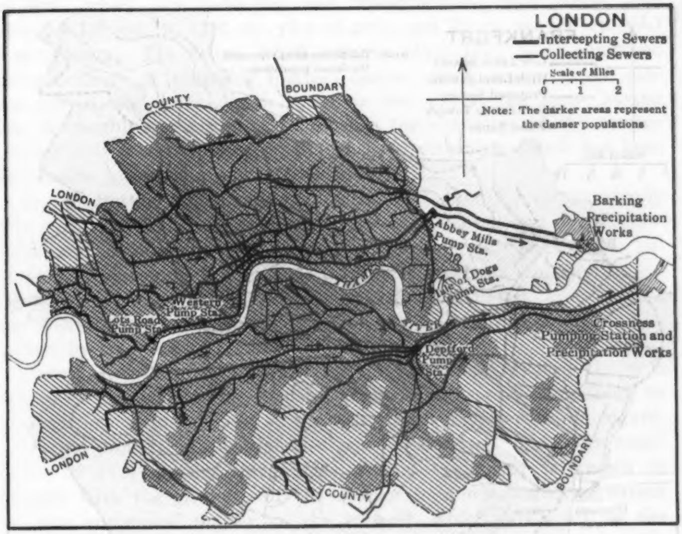

FIG. 40.

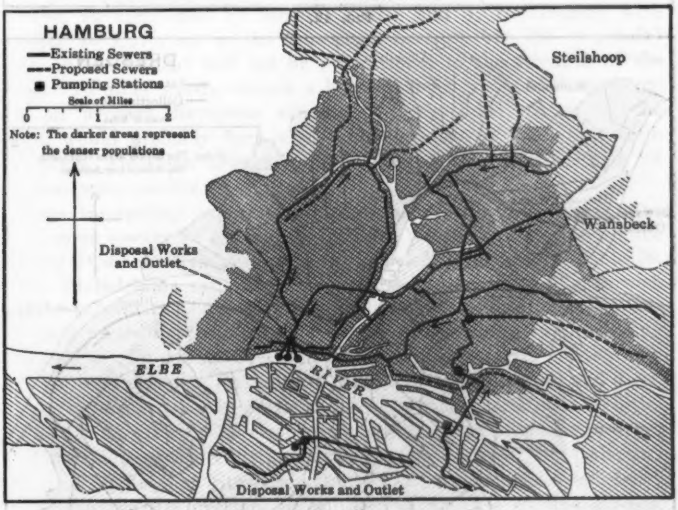

FIG. 41.

Mr. Allen.

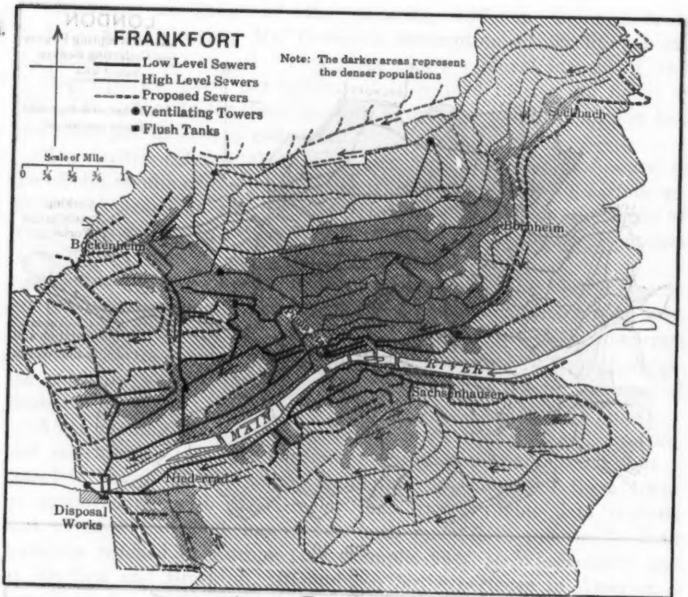

FIG. 42.

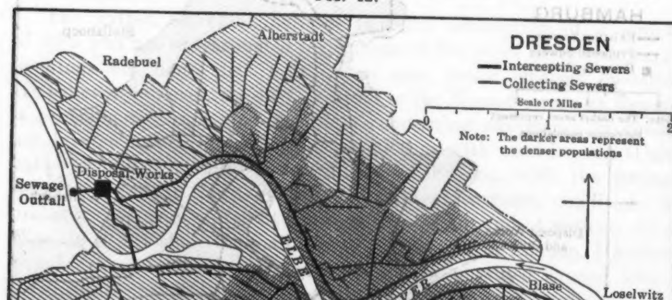

FIG. 43.

per day, which are recovered and manufactured into steam pipe covering, 150 cu. m. (195 cu. yd.) of grit, and 30 cu. m. (39 cu. yd.) of screenings. The latter, when seen in May, 1913, were composed almost entirely of vegetable matter: leaves, straw, orange peels, cabbage leaves, and sticks, with more or less paper, the feces having been thoroughly comminuted. Probably the character of the sewage received here is also influenced by the local screening which has been described so interestingly by Mr. Granbery. Mr. Allen.

At the Boston and London outfalls, also, there is comparatively little visible evidence of fecal matter. As pointed out by Messrs. Potter and Hammond, as a sewage becomes septic, the relative advantages of screening are lessened. In some cases, as with the Elberfeld-Barmen plant, the quantity of screenings removed, even after a long travel, may be large, due perhaps to the inhibition of decomposition by trade wastes.

Messrs. Kershaw, D'Olier, Potter, and Jennings regard the percentage of removal as of doubtful importance, stress being laid on the condition of the effluent as the real criterion of screening, regardless of the proportion of suspended matter removed. This test would appear to have decided advantages, for the efficiency is known to fluctuate with the strength of the sewage to a much greater extent than the suspended matter in the effluent, which, after all, is the important consideration. In comparing the results obtained by different plants, the strength of the sewage should, of course, be kept in mind, as mentioned by Messrs. Pearse and J. H. Gregory with reference to European *versus* American sewages, but the suspended matter in the effluent will not be proportional to the strength of the sewage. In other words, though a higher efficiency would be expected from an average European sewage, the suspended matter in the effluent would probably vary to a much less extent from that from an average American sewage, provided the screens and methods of operation were similar.

Some interesting experiments were made with different screens on the same sewage in Chicago by Messrs. Wisner and Pearse for the Sanitary District. The results were, in brief, as shown in Table 30.

The Philadelphia experiments described by Mr. Stevenson furnish valuable information of practical application, confirming what has been said regarding the favorable results of screening in preventing the clogging of nozzles, also on the favorable character of the subsequently settled sludge for rapid drying; Messrs. Hammond and J. H. Gregory mention its use in preventing scum formation; Messrs. Whittemore and J. H. Gregory on the consequent reduction of sludge storage; Mr. Whittemore of the reduction of the area of sludge-drying beds; and Messrs. Whipple and Stevenson point out the increased efficiency obtained in the disinfection of effluents by the use of fine screens.

Mr. Allen. These are all important considerations favoring screening, based on actual experience.

TABLE 30.

Size of slot, in inches	1.00 by 0.0625	0.50 by 0.032	0.050 by 0.025
Average reduction of suspended matter	14%	20%	25%
Time required to reach loss of head of 1.5 ft. at rate of 11 100 gals. per sq. ft. daily	28 min. 45 sec.	9 min. 8 sec.	7 min. 14 sec.

Meshes per linear inch	6	10	16	20	24	30	40	60	80	100
Average reduction of suspended matter	13%	10%	17%	16%	21%	20%	26%	28%	33%	29%

The method of estimating efficiencies from the dry material in the screenings and the suspended matter in the effluent, suggested by Mr. Kershaw and used by Messrs. Pearse and Greeley, appears to the writer to have distinct advantages in avoiding erratic results due to large masses of suspended matter included in, or omitted from, the sample. The writer is not an analyst, and does not feel competent to express a more definite opinion on this procedure, but if it should prove that results heretofore reported are unreliable for this cause he is most anxious to be made aware of it. The drop in efficiency at Dresden from 66.4% in 1912-13 to 30.95 in 1913-14, as given in Tables 9 and 10, was attributed mainly to the mode of sampling.

Several of those discussing the paper lay stress on the desirability of arriving at some generally acceptable method of sampling, and of estimating screen efficiencies, mentioning at the same time the great difficulty in doing this on account of the number and uncertainty of the factors involved.

Messrs. Franze, Schaefer, and Kershaw point out that screening has little effect in reducing putrescibility and Messrs. Soper and Hering that its effect on the reduction of turbidity is negligible. These statements are very true, and should be kept in mind in selecting a method of treatment. In fact, as mentioned by the writer and by Mr. Potter, the colloidal matter may be increased by screening. On the other hand, screening removes the more apparent sources of pollution and, as mentioned by Messrs. Hering and Hammond, permits of a lower ratio of dilution of the effluent without offense.

Mr. Potter is of the opinion that the increase of solids in solution, as computed for Gleiwitz, is in error. If the dried residue after filtration, as stated in Table 4 (which represents the solids in solution), is increased by screening from 1 237.2 to 1 260.4, this 1.9% of suspended solids is correct and the writer is unable to see how any qualifying effect due to stable solids in solution is admissible. The particular figure in any one case is of little importance, emphasis being placed on the fact that under certain conditions a portion of the

suspended solids may become colloidal or pass into solution in the process of screening. In fact, the writer agrees with Mr. Whittemore. that precise determinations are so uncertain that it is an unnecessary refinement to express percentage efficiencies in less than whole numbers. Mr. Allen.

Mr. Pearse says:

"Comparative results on the same sewage do not indicate that screening, even with a 30-mesh screen, is the equivalent of thorough sedimentation, nor within 50 per cent."

On close inspection, the writer fails to see where any such assumption was made. On page 883 he expresses his belief that the time has come "when screening may fairly be compared with tank treatment", but, that he did not consider them equivalent is evident from his statement on page 946:

"* * * the best fine-screening will compare well with such tank treatment as may usually be looked for: That is, from 30 to 50% of the suspended solids may be removed by fine screens, as compared with 50 to 65% by sedimentation."

Although the writer believes that the former figures are higher than it would be wise to assume in planning new work, they still leave a fair margin between the best screening and sedimentation, as ordinarily carried out.

Assuming 1.9 cu. yd. of screenings containing 375 lb. of dry substance per million gallons of sewage, as deduced from Table 11. and 1 527.5 lb. of dry matter per million gallons of sewage, as found at Providence with plain sedimentation, Mr. Kuichling has estimated a probable removal by fine screens of 46.65% of the removal by settling in tanks.

Mr. Pearse infers that, in comparing screening with tank treatment, the writer has considered openings as large as 0.6 in., whereas these are specifically stated as "not more than 0.10 in. ($2\frac{1}{2}$ mm.) in size".

With regard to the whole question of screen efficiency and the relative value of screening *versus* settling, the writer desires to say that he has no axe to grind, in either connection, and can express his attitude no better than by repeating the following statements from the paper:

"* * * it is the writer's opinion that fine-screening will be adopted in the future by many towns situated on bodies of water which are capable of assimilating the effluent; possibly, also, as a preliminary process to tank treatment, filtration, and disinfection. On the other hand, the relatively high cost of attendance and the probable lack of a market for the screenings, when compared with conditions abroad. will probably serve to prevent the marked increase in their use in the United States which has been experienced in Germany."

The discussion by Mr. Soper furnishes an excellent summary of the relative advantages of each method of treatment.

Mr. Allen. The writer is sorry that more data regarding screen efficiencies have not been forthcoming, and realizes more than ever that additional information must be had before any figure can be viewed as other than tentative.

There is one more point: In any consideration of fine-screening it would seem proper to include the auxiliary process of grit removal, which may or may not be required. In other words, in a comparison of plants, these should be considered complete, with all the accompanying features depending thereon, not only grit removal, but power, disposal of screenings, and attendance. This view is also expressed by Mr. Soper.

Mr. Kershaw mentions the deposit of grit as possible where the current above the screen is retarded by the screen itself. This is important. The velocity above the screen should be sufficient and that through the screen not so great as to force the larger solids through. With the U-shaped channels of the drum and Riensch-Wurl screens, and the method of scooping up from the bottom with the shovel-vane screen the likelihood of deposits near the screen is reduced. As a general proposition fine screens should have grit chambers placed above them for their protection.

Mr. Kuichling's suggestion that screens be cleaned by suction is one which certainly deserves serious consideration. If worked out so as to be a mechanical and economic success it would appear to be ideal. To the writer, however, there seems to be little objection to the methods used at Hamburg, Dresden, and Frankfort, so far as odors, spray, or other sanitary considerations are concerned.

The question of head consumed has been mentioned by Messrs. Greeley, Kuichling, Pitkethly, and Riedel. Answering Mr. Kuichling's question as to the coefficient of friction, 0.4, the writer quotes as follows from a letter of March 2d, 1914, from Mr. H. H. Stevens, of the Sanitation Corporation, Philadelphia.

"Mr. Wurl's method of figuring the loss of head is to consider about one-fourth of the submerged area as being effective opening, and to assume one-half to two-thirds of this effective opening being covered by screenings in order to arrive at the true effective opening for passing the effluent through the screen, and then computes the loss of head on the basis of the formula,

$$H = \left(\frac{Q}{C \sqrt{2\,G} \times A} \right)^2,$$

using the coefficient, C, at a value of 0.4, which he claims has been proven by test."

He adds that, on account of the weaker American sewage, the Sanitation Corporation figures that about one-fourth of the effective opening will be covered by screenings. A coefficient of, say, 0.6, would

undoubtedly correspond more nearly with what meager data on small ajutages are available, but the use of a smaller coefficient is no doubt preferable in providing a greater factor of safety. The writer, however, is not responsible for the particular value mentioned. He regrets that he cannot give other figures for drop in head besides those already mentioned, except in the case of the Drum screen used in the Mainz experiments mentioned on page 916 in which this varied from 1.4 to 7.3 in. Mr. Allen.

Mr. Hammond and Mr. C. E. Gregory speak of the condition of the Thames, Elbe, Rhine, and other European rivers which receive sewage effluents from large populations without resort to the oxidizing processes often supposed to be necessary. At London and Glasgow the effluents from precipitation plants appear somewhat turbid if more than 1 or 2 ft. in depth, generally of a reddish-brown color, but without visible solids, grease, or odor. At Crossness (London), partly due to the brownish-green color of the river, the effluent could be followed for several hundred feet from the outlet, but no other sign of pollution was noted, such as sleek or floating solids.

At Frankfort, after fine screening and plain sedimentation, the effluent appeared to be free from solids, but was somewhat cloudy when of much depth. The raw sewage was dark grayish-brown.

The writer was unable to discover the effluent when passing near the three submerged outlets at Hamburg. Mr. Kuichling has stated:*

"The result has been entirely satisfactory, and it is now reported to be almost impossible to detect any evidence of sewage pollution in traversing the river with a vessel. The suspended matters in the sewage that pass through the screens seem to remain near the bottom, and to be finally carried out to sea by the current."

In spite of the coarseness of the screens used at Hamburg, the absence of floating matter is also confirmed by Mr. Hammond. It should be said, however, that the river itself is somewhat brownish and turbid. If clean and saline, the effluent would be much more apparent. According to H. N. Ogden, M. Am. Soc. C. E., the manager of the plant stated that "very seldom and only in the winter could anything in the nature of a grease film upon the river surface near the outlet be detected".†

Experiments made by the Metropolitan Sewerage Commission of New York, by discharging sewage, or fresh water colored by a strong dye, at different depths below the surface illustrated in a very convincing manner the influence of salinity on the appearance of sewage effluent at the surface. With fresh river water, as at Hamburg and Washington, there is much less tendency to rise, and diffusion is more prompt; but with a saline water such as that of Boston (Nut Island outlet)

* "Notes on Sewage Disposal," Rochester, 1912, p. 19.

† *Engineering News*, October 13th, 1910, p. 386.

Mr. Allen. or New York Harbor, the sewage rises promptly to the surface, where it spreads out in a thin sheet and is made more apparent by the saponification of the grease. This difference between river and ocean water with respect to sewage effluents should always be kept in mind.

Mr. Kershaw's inquiry regarding stream flow and dilution is answered in part by the following notes.

At Barking and Crossness some 400 000 000 gal. of London's sewage per day, from a population of probably 6 300 000, is discharged into the Thames after being treated by chemical precipitation. The river here has a tidal range of some 20 ft. and a net flow toward the sea of about 190 000 000 cu. ft. per day.* This furnishes a dilution, in addition to that derived from the oscillation of the tide, of about 5.1 volumes, or 3.5 cu. ft. per sec. per 1 000 persons. The average salinity, as computed by the writer, is about 200 grains per Imperial gallon, which is equivalent to a mixture of 5 parts of river to 1 part of sea water.† Although there is a satisfactory absence of floating solids in the water, apprehension has been felt concerning the depletion of oxygen in the Thames near the outfall where, at times in summer, it is nearly complete.

At Frankfort, the River Main receives the tank effluent from a population of 420 000. This effluent contains only 20% of the suspended matter in the raw sewage, half of which is non-sedimentable. The latter is variously stated as 411 and 536 mg. per liter = parts per million.‡ The flow of the Main and the corresponding volumes of dilution are as follows:

During low	water =	6 400 000 cu. m.	per day =	128 volumes	
" mean	"	= 15 000 000 "	" =	302	"
" high	"	= 83 800 000 "	" =	1 676	"

At Dresden the average flow of the Elbe is somewhat less than 79 cu. m. or 2 790 cu. ft. per sec. The water contains 6.2 tons of solids per 1 000 000 cu. ft., and the sewage 37.4 tons. After receiving the effluent the river water contains 7.8 tons per 1 000 000 cu. ft. It is expected, when the population reaches 800 000, that to 293 cu. ft. of solids brought down by the stream, 86 cu. ft. will be added by the sewage effluent.

At Hamburg, 53 000 000 gal. of screened effluent per day from about 1 000 000 inhabitants are discharged in the bottom of the Elbe, the low-water discharge of which is here 5 200 cu. ft. per sec. or 5.2 cu. ft. per sec. per 1 000 population, and the mean discharge 23 000 cu. ft. per sec. or 23 cu. ft. per sec. per 1 000 population. Therefore, there is a dilution of 63 volumes at low water and of 280 volumes at times of mean flow.

* "Movement of the Water in a Tidal River," W. Cawthorne Unwin, 1883.

† Report on Thames Conservancy by Medical Officer of Health, 1907.

‡ Uhlfelder and Tillmans, "Wasser und Abwasser," Vol. I, No. 7.

The automatic control of speed which Mr. Kuichling has mentioned as so desirable with a variable flow, is accomplished with the Carshalton screen, the motion of which depends on the undershot wheel actuated by the flow of sewage. In the other types of screen this may be accomplished by auxiliary mechanism if desired, although there is a certain compensation in most screens by the increased area of screen surface submerged with an increasing flow. Mr. Allen.

The writer admits that in the paper the subject of the disposal of screenings was not given the consideration it deserved, and is glad to see that it is taken up by several of those who contributed to the discussion.

The quantities obtained at Hamburg, Frankfort, and Dresden, as given by Mr. Greeley, differ materially from those in the paper, namely:

		Hamburg.	Frankfort.	Dresden.
Allen......	Per day	18 cu. yd.	50 tons	25.6 cu. yd.
Greeley....	Per day	26 cu. yd.	20-26 cu. yd.	13-16 cu. yd.

These differences may be due to errors in information furnished, or to altered conditions, but with regard to the writer's figures he would say that that for Hamburg was obtained from Baudirektor Sperber in a letter of April 2d, 1914. That for Frankfort was obtained verbally at the works, was possibly misinterpreted, and is believed to be incorrect. A preferable figure is 17 cu. yd., being the average daily output in 1913, and obtained from Stadtbaurat Franze under date of December 30th, 1914. The average volume obtained at Dresden was given for 1913 by Stadtbaurat Fleck in a letter of April 9th, 1914.

The possibility of a nuisance from odors with the storage of screenings mentioned by the writer, has been spoken of by Messrs D'Olier, Potter, Greeley, J. H. Gregory, and Soper. In this respect, well-digested sludge has a decided advantage, as mentioned by Messrs. Hammond and Potter, but the satisfactory disposal of screenings is believed by the writer to be a simpler matter than that of ordinary settled sludge or that from chemical precipitation. It is essential that means be provided for prompt removal and disposal in conjunction with screening plants, but there should be no serious difficulty in doing this, by dewatering and burning, or otherwise. The odor of screenings stored for several days resembles that of a pig-stye, and would, under certain conditions, be objectionable at a distance of several hundred feet. At Cologne, the screenings are stored in bins about 100 ft. long, 11 ft. wide, and 4½ ft. deep. Two or three of these were filled when visited in June, 1913. The daily output is about 21 cu. yd. per day. Flies were abundant, and the odor was very unpleasant. Under certain conditions, this was said to be noticeable at a distance of nearly 1¼ miles; at Dresden, ¼ mile. The screenings are valued as a fertilizer, but their storage is a serious nuisance. The cost of disposal, naturally, should be included with the other costs of operation, in comparing different schemes.

Mr. Allen.

Mr. Kershaw mentions the availability of screenings as a manure. This may be practicable in some instances, as the screenings undoubtedly have a fertilizing value, but there should be some assurance that they will be removed regularly and not stored where likely to give offense. An analysis of the Hamburg screenings is given in Table 1. The following is given by Tillmans* for Frankfort:

	Grit.	Screenings.
Moisture	60-70%	80%
Of the dried material:		
Nitrogen, N	0.12%	0.82%
Phosphoric acid, P_2O_5	1.01%	1.57%
Potash, K_2O	0.12%	0.40%

At Wiesbaden the screenings are stored under a shed and covered with sweepings, where they remain until taken by neighboring farmers. By some such way or by composting, the danger of nuisance may be reduced.

In Germany it may be said that the utilization of screenings as a fertilizer is the ruling custom, but where the percentage of fats is as high as at Dresden, where it is said to be 18%, there appears to be a promise of profit in its recovery. Here the screenings are first drained, shrinking to one-third their original volume, and have recently been heated in a rotary dryer, treated chemically to remove the grease, and the products sold at a net profit of $15 000 per annum.

The cost of the Frankfort screening plant, recently received from Stadtbaurat Franze, is as follows:

Grit elevator and motor	6 600 Marks =	$1 570.80
Three automatic screens, with motor and appurtenances	21 600 Marks =	5 140.80
Apparatus for removal	4 800 Marks =	1 142.40
Total	33 000 Marks =	$7 854.00

Disposal by incineration, with refuse where practicable, as at Frankfort, and mentioned by Messrs. Franze and Schaefer, has the advantage of being inoffensive and practically without cost.

Costs of installation and operation of German plants, as mentioned by Messrs. Cohen and Potter, are not directly applicable to American conditions, but are valuable in matters of comparison between each other and, after allowing for differences of ruling wages, etc., as a guide for projected work elsewhere.

In reply to Mr. Kershaw, the figures for cost of operation that have been given do not include charges for interest or amortization. In a comparison of different schemes these, of course, should be included.

* "Water Supply and Sewage Disposal."

Regarding the very high cost given for the Dresden plant ($2 150 000), it should be explained that this covers the cost of grit chamber, elevators, administration buildings, workshop, heating, power and electric lighting plants, and all appurtenances besides that of the screens alone.* Mr. Allen.

Mr. Pearse asks why 0.6 in. was taken as the dividing line between coarse and fine screens. The precise limit is quite arbitrary. The writer agrees that, say, 0.2 or 0.15 in. would for many reasons be better. In the paper 0.6 in. was adopted so that the Hamburg screens might be included in the comparison, both on account of the importance of these screens and of the band type of screen in general, others of which are of much finer opening.

Messrs. Hering and Cohen mention the use of screens before centrifugal pumps. Perhaps a more positive advantage would follow in their use before triplex pumps, which, as a rule, operate at higher efficiencies and are more likely than centrifugal pumps to be injured by coarse solids.

Regarding the applicability of fine screens to New York City, referred to by Mr. D'Olier, the conditions at different outlets and suggested points of disposal vary very greatly. For some of these a high degree of treatment will probably be required eventually, for others tank treatment will be sufficient, and it is probable that there are yet others where fine-screening will prove most advantageous. In parts of Manhattan the sewage is (for American sewage) very strong, besides being very fresh, both of which considerations favor this method of treatment.

As to the type of screen, the writer feels that, from lack of familiarity, the Windschild and Geiger screens have not received the recognition in the discussion that they deserve. Probably no one type can be pronounced as best; the selection should depend on local conditions and prices.

In closing, the writer thinks it appropriate to mention the work done by the late Emil Kuichling, M. Am. Soc. C. E., in collecting European data on fine-screening. This was done with his customary thoroughness, and presented in a practicable form for the use of American engineers at a time when fine-screening was scarcely known here. With his keen insight, he realized both the possibilities and the difficulties to be overcome in the use of fine screens, and was one of the first to advocate their adoption in America. Much of the writer's interest and information regarding them is due to Mr. Kuichling, who was always ready and willing to spare his valuable time to discuss the subject, and it creates mingled feelings of regret and appreciation to note that his discussion of this paper was the last work done by him.

* *Municipal Journal*, December 28th, 1911.

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Paper No. 1327

REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS*

BY HARRISON S. TAFT, ESQ.

WITH DISCUSSION BY MESSRS. J. H. H. MUIRHEAD, W. J. BARNEY, EUGENE W. STERN, R. D. COOMBS, JOEL J. PEMOFF, WILLIAM GOLDSMITH, DEWITT C. WEBB, JOSHUA F. RAMSBOTHAM, CHANDLER DAVIS, E. G. WALKER, W. J. DOUGLAS, AND HARRISON S. TAFT.

SYNOPSIS.

During the past few years considerable discussion has taken place among engineers and chemists as to the feasibility of using cement in sea water, and the practicability and commercial success of reinforced concrete docks. In the United States the construction of reinforced concrete docks is in a very embryonic state, and the use of cement in structures standing in sea water, on the part of American engineers, has not always been successful. On the other hand, concrete has been used successfully for more than 50 years in Europe for structures exposed to the action of salt water, and English engineers have been building reinforced concrete docks for about 20 years.

Part I of this paper contains a brief outline of the reinforced concrete dock problem as developed in Europe and Asia, the work in each country being reviewed. Part II reviews the art as carried out in the United States, both in sea water and fresh water. These two parts cover

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* Presented at the meeting of May 20th, 1914.

descriptions of reinforced concrete docks, as well as construction features of special interest.

Part III takes up the subject of failures in past uses of cement in sea water in America, points out the reasons therefor, and brings out the fundamental principles covering the successful use of concrete in structures exposed to salt water and frost action. A foot-note calls attention to the chemistry of salt water cement, and the use of concrete in sea water, in order to bring out discussion on this phase of the problem.

Part IV goes briefly into the question of the initial cost of reinforced concrete docks, as well as the cost of annual repairs.

Part V is a review of opinions of foreign expert dock engineers on the use of concrete in sea water and on the practicability of reinforced concrete docks; there is also a short review of the use of concrete in foreign breakwaters, closing with a few remarks as to the reasons for the success of foreign engineers in using concrete in sea water.

Part VI contains the writer's conclusions, wherein he cites the relative positions of English and American engineers in respect to the reinforced concrete dock question, and a more general use of concrete. He also speaks of progressive engineers possessed with the determination and courage to develop new ideas and uses for concrete, with due consideration as to previous failures, while conservative engineers still cling to old-time, long-established practices.

In the closing paragraph the writer calls the engineer's attention to the necessity of special training and knowledge for the successful execution of concrete structures in salt water.

The object of the paper is to make known to the Engineering Profession the exact up-to-date situation with respect to the development of this problem, the world over, the feasibility of using cement in sea water, and the practicability of building reinforced concrete docks.

Introduction.

In undertaking a study of the reinforced concrete dock problem, the subject has so many phases as to require a number of chapters to cover fully all the various ramifications into which it leads. From a world point of view, reinforced concrete docks are not a novelty. On the other hand, their practicability and commercial economy have perhaps as yet to be proven, especially in America. It is true that there

are many reinforced concrete docks in successful operation in foreign countries, the most extensive development being found in England. As their size and number are steadily increasing, it would appear that this class of dock construction has proved its worth, and sooner or later will become the standard type where a structure of a permanent nature is desired.

As reinforced concrete docks are somewhat of a novelty in the United States, it is perhaps necessary for engineers to seek the experience of foreign countries in order to obtain the necessary data on which to base their conclusions as regards the feasibility or otherwise of this type. Thus it is eminently fitting to investigate first what has been done in various parts of the world in the way of building such docks and the success accompanying such undertakings, and then discuss other phases of the problem, as to what precautions, etc., are necessary to render them a commercial, practical, and engineering success.

Part I.—Reinforced Concrete Docks in Foreign Harbors.

England.

In building reinforced concrete docks and other concrete structures in sea or fresh water, it is only natural that a forested country should be the last to take up the devolopment of such a type of dock construction. Consequently, America has been far in the rear in regard to this question, as compared with what other and older countries have accomplished. In England, France, Germany, Italy, and other European countries, in Australia, and in Asia, as well as in certain South American countries, concrete sea-walls, breakwaters, dry docks, piers, trestles, coaling stations, etc., in salt water, have been in existence for years. In looking up past undertakings along these lines, the writer was amazed to learn of the large number of reinforced concrete docks that have been built in foreign countries and the great success that has apparently been attained with them.

Southampton.—One of the most noted developments in reinforced concrete construction, as applied to harbor and dock development, is that of the London and Southwestern Railway Terminals at Southampton, England. The first and most prominent reinforced concrete structure in connection with this terminal was a coal barge jetty, 300 ft. long and 20 ft. wide, built in 1904, on the Hennebique system of driven concrete piles. The piles are about 44 ft. long, standing in 29

ft. of water at high tide, the rise and fall of the tide being about 13 ft. Each pile carries a maximum load of 17 tons.

This structure carries a very heavy traveling coal-hoisting apparatus for unloading coal from large vessels docked on one side, into harbor barges or scows which lie on the other side. Thus the jetty is subjected to constant blows from both sides, in addition to the heavy vibration due to the traveling machinery it supports.

In speaking of this jetty, Mr. Francis E. Wentworth-Sheilds, Dock Engineer for the London and Southwestern Railroad, says:

"Though the impacts from the vessels and scows cause this whole jetty to sway, there seem to have been no signs at the end of the first 2½ years of its existence of any of the concrete peeling off."

During recent years this dock or jetty is said to have shown some signs of deterioration, due to the vibration of the heavy machinery traveling along it, the supposition being that it was built too light in the first place to absorb the heavy vibrations to which it has been subjected. Still, the dock is said to have considerable elastictity. In one instance it was in heavy collision with a steamer, two piles and the beams they carried being broken. The dock was effectively repaired, but perhaps with some difficulty, though it is claimed that the repairs were easily accomplished.

The same railroad company has built several other reinforced concrete pile docks at Southampton, designed to carry the same heavy deck loads as the coaling jetty, but without the heavy vibration to which this jetty is subjected. Though built on the same system of construction as the coal jetty, the latter docks have shown no signs of wear or deterioration. It is said, on the best of authority, that they have cost nothing to date for maintenance.

The largest of these is the extension, on the Itchen Front, of "The Empress Dock", a widening of the so-called "Old Extension Quay" by a reinforced concrete pile structure, 50 ft. wide and about 1 300 ft. long, parallel to and securely dovetailed into the old quay wall. This widening dock is built of complete concrete bents, along the tops of which are steel deck-beams or stringers, which in turn are covered with 4 in. of wood, a wooden block pavement being used for the wearing surface. The depth of water at low tide at the face of this structure is 35 ft., which, with a 13-ft. rise and fall of the tide, gives a depth of 48 ft. at high tide.

One of the finest cold storage and cattle stations in existence was built at Southampton in 1905 to accommodate the foreign cattle trade. The landing stage or jetty of this station is a reinforced concrete structure, 200 ft. long and 38 ft. wide, connected with the main land by two runways, 142 ft. long and 15 ft. wide.

On the opposite side of Southampton Harbor, at Woolston, on the Itchen, there is a reinforced concrete landing dock, 136 ft. long and 100 ft. wide, built in 1899 on the Hennebique system. This was the pioneer of reinforced concrete dock construction in Southampton waters. Up to date, this dock has cost practically nothing for repairs, except for damages due to the fact that it was rammed or otherwise damaged by a large steamer; it is in excellent condition at present. The cost of making the repairs is said to have been very small.

Up to the present time, it appears that at least six reinforced concrete docks, jetties, or quays, have been built in Southampton waters.

In building one of the Southampton docks, it is stated that some of the concrete piles were sprung out of line in driving them. A prominent American engineer reports that he saw a number of piles from 1 to 2 ft. out of line, but that they showed no signs of cracks. It has been stated that, in handling Chenoweth concrete piles, up to a length of 61 ft., and 13 in. in diameter, they were rolled about like wooden piles, at times having quite a spring in them. Under this treatment they showed a remarkable degree of elasticity and no signs of cracking.

Bending of Piles.—In discussing the bending of concrete piles, it is of interest to note a series of tests made on a hollow telephone pole in Fulham, London, England, in 1911. The pole was 44½ ft. long, 17 by 17 in. at its base (outside dimensions), tapering to 8 by 8 in. at its head. The thickness of the shell was 2 in., making the inside dimensions of the pole 13 by 13 in. at the base and 4 by 4 in. at the top. The vertical reinforcement consisted of 248 $\frac{3}{16}$-in. rods of high-tension steel, the ultimate tensional stress being from 80 000 to 85 000 lb.; 56 rods were grouped at each corner, 6 rods being spaced evenly on each of the four sides of the pole. The area of the concrete at the base was 106 sq. in., and the area of the steel 6.85 sq. in., a ratio of 0.0445. In making the test, the pole was set in 5½ ft. of massed concrete, with the pulling rope attached to its upper end.

In Table 1 is recorded the pull, in pounds, the deflection, and the permanent set after the loads had been released.

After applying the 6 000 lb., the test was discontinued for 3 days, and then the load was slowly increased to 9 200 lb., which gave a deflection of 66 in. and a permanent set of 21 in., with "cracks on tension side and permanent cracks more numerous and pronounced." There was no sign of failure on the compression side of the pole.

TABLE 1.—TEST OF A HOLLOW CONCRETE POLE AT FULHAM, ENGLAND.

Loads, in pounds.	Deflection, in inches.	Permanent set, in inches.
300	¾	None that could be observed.
500	2	
1 000	4¼	
1 250	5	
1 500	6¼	
1 750	8	
2 000	9	
2 250	10½	
2 500	12	¼
2 750	14	½
3 000	14¾	⅝
3 550	19½	¾
3 100	16½	½
3 500	18¾	⅝
4 000	22	¾
4 500	25⅛	1⅛
5 000	29	1¾
Load gradually increased to 5 750	34	Pulling wire broke. Pole flew back, vibrated short time and came to rest in vertical position with no permanent set.
Load gradually worked up to 6 000	37⅛	2 in. immediately after release of load. After interval of 1⅛ hours, set was reduced to 11/16 in.

Add to the above loads 100 lb. for weight of wire and recording apparatus, etc.

Notes on Behavior of Pile.—At 3 350 lb., slight hair cracks appeared on tension side at about 5 ft. 6 in. above foundation.

At 3 600 lb., slight hair cracks appeared at regular intervals about 6 to 9 in. apart.

At 4 100 lb., slight shear cracks appeared, starting at 33 in. above foundation and extending vertically, at a distance of 2 in. from tension side, for some 3½ ft. in length.

At 4 600 lb., hair cracks occurred regularly, 3 or 4 in. apart on tension side.

At 5 100 lb., cracks on tension side were noticed to be traveling across the sides of the pole to within 6 in. of compression surface.

At 6 100 lb., the shear cracks were more pronounced. The hair cracks on tension side were about 1 in. apart, but no signs of failure appeared on compression side.

The load was again applied. When a deflection of 73 in. was obtained, signs of failure appeared for the first time on the compression side, and the pole failed, but no record of the amount of the pull was obtained. The final examination showed that the pile failed equally for its entire length on the shear and tension sides, but without any local weakness. The compression side failed from the base up for a distance of about 2 ft. The bending moment at the base of the pile was reported to be 4 863 936 in-lb., a most remarkable test, which seems to substantiate the experience quoted above.

Liverpool.—The Mersey Docks and Harbor Board appears to have made a very limited use of reinforced concrete in its latest port developments, in spite of the extensive tidal and graving docks of concrete constructed at Liverpool. As a matter of fact, most of its first so-called reinforced concrete docks were in reality semi-concrete structures, that is, wooden piles carrying a concrete deck system. The oldest of these structures, the Cattle Wharf, Prince's Stage, was built in 1899-1901 on greenheart piles, with the usual Hennebique system of concrete deck-beams and slab. In addition to the Cattle Wharf, there are two other dock-quays in Liverpool, built on the same system, designed for a load of 3 tons per sq. yd., with a test load of 4½ tons.

Another of Liverpool's reinforced concrete docks is the Prince's Dock, West Quay, completed in 1905. This structure was designed to carry a super-load of 3 tons per sq. yd., or about 66 tons per pile. The piles are of concrete, 16 in. square, spaced 15 ft. 9 in. from center to center, across the dock, and 12 ft. 6 in. from center to center, longitudinally, and resting on rock.

At the west end of Liverpool's dock system is the Brocklebank Dock, 226 ft. long and 64 ft. wide, a reinforced structure built in 1908 on the Hennebique system, and designed to carry the same loading as the Prince's Dock. The test load applied was 9 tons per sq. yd. The piles of this dock are 20 in. square, and had to sustain a load of about 95 tons each, when the test load was applied.

During the first year of its existence there was considerable trouble with the Cattle Wharf and other semi-concrete structures, due to the permeability of the deck-slab part of the structure. The dampness from the salt water entered the permeable concrete and attacked the steel reinforcement on the under side of the slab, causing pieces of concrete to fall off. The defect was overcome by applying a heavy coating of cement to the under side. The upper side of the deck-slab, which is washed daily, shows no signs of deterioration. This emphasizes the fact that great care must be taken in placing concrete in structures standing over or in salt water; there must be the closest inspection during their construction. As a result of this trouble, it appears that the steel reinforcement must be kept farther from the surface of the concrete in salt water concrete structures than in ordinary work. Whereas, at first a minimum of 1½ in. was supposed to be sufficient, the engineers of the London and Southwestern Railway have found this to be not enough, and are now specifying a minimum of 2 in.

for their new structures. Another difficulty encountered was the numerous joints in the Liverpool system of dock construction. This appears to have been overcome by cleaning out the joints and filling them with a rich cement grout, with such success that the docks are reported to be in very satisfactory condition at the present time.

Thames River, Etc.—Within the bounds of the "Port of London Authority", and at other places on the Thames, many reinforced concrete docks or quays have been constructed. One of the most prominent of these is the Thames Haven Jetty Head, near the mouth of the river. This structure was built in 1908, for the berthing of large vessels, and is 136 ft. long and 32 ft. wide. It is supported by 19 columns or piers, 5 ft. in diameter up to low-water mark but of oval shape, 5 by 2½ ft. above. Each column rests on two 15-in. concrete piles of octagonal section, 45 ft. long, with a penetration of from 18 to 20 ft. After these concrete piles had been driven to refusal, a temporary iron caisson was put down over them. The reinforcing hoops were then lowered into place around the two piles and the caisson was filled with concrete. The caisson was afterward removed, being built in half sections.

Farther up the Thames, 20 miles from its mouth, is the Purfleet coaling jetty, built in 1904, a reinforced concrete structure, 250 ft. long and 34 ft. wide, carrying two heavy traveling cranes. The concrete piles are 14 in. square, from 40 to 50 ft. long, with from 15 to 18 ft. of penetration. The deck-slab is 5 in. thick. The jetty is connected with the mainland by a reinforced concrete approach, over which cars are run to the jetty itself. On one occasion this jetty was rammed by an 8 000-ton steamer, eight piles and 20 ft. of the decking being damaged. That the damage was not more extensive is said to have been due to the firmness of the horizontal decking. In repairing the dock it was necessary to withdraw the eight broken piles and drive new ones. In conducting this repair work, it was especially noticed that the steel reinforcing bars of the original structure showed no signs of rust, though it is well to note that the water at this point of the Thames may be brackish, and not real sea water. The repairs were efficiently made, and the quay is still doing duty.

Another interesting reinforced concrete coaling dock is at Dagenham-Essex-on-the-Thames, 10 miles below London, built in 1901. It is 780 ft. long and 35 ft. wide, and carries eight heavy traveling cranes, weighing 60 tons each, besides a railroad track. Each supporting col-

umn consists of three concrete piles encased in a concrete cylinder filled with concrete. Though it may seem a trifle odd, a careful study of photographs of this structure indicates that the tops of the columns are made with "capitals"—a well-proportioned and artistic structure as regards concrete dock work. A similar structure, known as the Prince of Wales Pier, was built in 1903 at Falmouth.

The Thames Iron Works and Shipbuilding Company has constructed at Dagenham a very substantial reinforced concrete dock, designed to carry a concentrated load of 60 tons, for the berthing and fitting out of dreadnaught battleships. In the construction of this dock, the Williams type of concrete piles was used for the first time. This type consists of an I-beam surrounded with $\frac{3}{16}$-in. wire, 12 in. from center to center, the whole being encased in concrete, with special provision at each end to take up the reaction of the driving.

In the same vicinity is the Hornchunk Dock and approaches, built in 1906. The main structure is 400 ft. long and 24 ft. wide, the approach being 280 ft. long and 16 ft. wide; both were built on the Hennebique system. The concrete pile bents are 14 ft. from center to center, braced diagonally, and support a concrete beam and deck-slab system. Along the water-front side of this dock and its approach, a concrete curtain-wall was built between the piles. This curtain-wall in turn is protected by a wooden fender system. The plans seem to indicate that this structure was designed to carry a 27-ton crane in addition to a 27-ton locomotive.

A reinforced concrete coal dock was built in 1906 at Rochester, on the Medway, a river flowing into the estuary of the Thames. This dock consists of a main water-front section, 32½ ft. wide and 340 ft. long, connected with the land by two concrete approaches, 100 ft. and 180 ft. in length, respectively. It was built on the Hennebique system, and carries a heavy traveling crane in addition to two railroad tracks. This was one of the first large undertakings in reinforced concrete dock construction on the Thames or its tributaries. At the time it was built it was looked on as the most important structure of its kind in England.

Though the exact number of reinforced concrete docks now in existence on the Thames or its tributaries is perhaps unattainable, it appears that they are more numerous in those waters than in any other part of England.

General.—Perhaps the largest reinforced concrete dock constructed up to date in England is at Swansea, on the southwest coast of Wales. It is a Hennebique design, almost 2 000 ft. long, completed in 1908, the whole structure being built in the dry. Some of its columns are 3½ ft. square. As this structure is used as a coal loading quay, and stands in 40 ft. of water, it is subjected to heavy loads and severe lateral shocks.

At the naval dock yard at Rosyth, Firth of Forth, Scotland, is found one of the most unique and interesting reinforced concrete docks or piers in existence. It is triangular in shape, 620 ft. long, has been finished recently, and forms the entrance pier to a tidal basin. The outer end consists of seven concrete monoliths with a mass concrete fill in the center, making a solid concrete head, 127 ft. long and 65 ft. wide, covering an area of some 8 300 sq. ft. The rest of the pier consists of twenty concrete monoliths, 25 by 30 ft., supporting a reinforced concrete structure consisting of girders, columns, arches, braces, and deck-beams. The upper decking consists of 3 by 10-in. creosoted wooden joists covered with a creosoted planking arranged so as to prevent any water from collecting thereon. It is a most massive and homogeneous structure, well capable of absorbing any heavy stresses it may receive from warships lying alongside.

In the Bristol Channel, at Clevedon, where the tide has a rise and fall of some 49 ft., there is another interesting reinforced concrete structure, in the nature of a landing stage. It is 95 ft. long and rests on 22 reinforced concrete piles. The piles, extending up to low-water mark only, were driven through the marl until they reached hard rock. On top of these piles the reinforced concrete landing stage was erected, a structure consisting of columns, beams, bracing, and four different decks or landings.

A reinforced concrete dock of magnitude was recently constructed at Port Talbot. Being designed for coaling operations, it is subjected to heavy loads. The designed load was 850 lb. per sq. ft., the test load being 1 390 lb. per sq. ft., covering an area of 720 sq. ft. The outer row of piling consists of two 14-in. square piles encased in a concrete cylinder, 4 in. thick, and 4 ft. 6 in. in diameter. There are six such columns at the face of the dock. Each of the other two rows consists of eight 14-in. square piles. At this port there are also a number of similar structures, the first having been built in 1907.

It is of interest to note that one of the reasons which influenced the engineers in adopting reinforced concrete for the coaling jetties and wharves at Port Talbot Docks was an extensive fire in one of the docks, due to fuel oil which had escaped from one of the vessels. The oil was ignited through carelessness, and caused a very intense blaze over an area of about 250 by 50 ft., though, fortunately, the oil did not spread out under the wooden structures. With the world-wide use of oil as a fuel for the merchant marine, it is well to consider this danger in American wooden dock structures.

Although the Port Talbot Railroad and Dock Company was among the first to adopt reinforced concrete for wharf construction in England, and so was adversely criticized "for using an alleged untried material in such types of work", the results obtained have fully justified this radical departure from what was at that time the prevailing practice the world over.

Scattered all through other English ports are reinforced concrete docks of various sizes.

At Fleetwood, from 35 to 40 miles north of Liverpool, a reinforced concrete fish and coaling dock of considerable magnitude was completed in 1911. The fish shed section is 1 330 ft. long and 26 ft. wide, that is, the reinforced concrete quay part; the filled-in land behind the quay makes the shed sections some 70 ft. wide in all. The coaling section, of similar construction and carrying a coal-loading traveling crane, is 680 ft. long and 26 ft. wide.

At Harwich, 40 miles northeast of the Lower Thames, is the Parkeston Quay, a reinforced concrete dock structure more than 1 000 ft. long and 51 ft. wide.

On the northeast coast of England, at Newcastle-on-Tyne, a reinforced concrete jetty wharf of extensive size lies along the water-front of a large turret shop.

The port authorities at Dundee and Aberdeen, Scotland, are gradually replacing their worn-out wooden dock structure with reinforced concrete.

On the north coast of Scotland, at Ackergill, stands a life-boat slipway, 194 ft. long, built of reinforced concrete, extending out from the rocky shore into the wide open sea. There are several similar structures in other parts of Great Britain.

A reinforced concrete dock was built in the Shetland Islands in 1910. The head of this dock is 80 by 24 ft., and is reached by a concrete approach, 113 ft. long.

In Cork Harbor and other Irish ports reinforced concrete docks of considerable size have been and are being built.

At Portsmouth, Plymouth, and Cardiff are found the first reinforced concrete docks constructed in England, having been built previous to 1906. They are small, and are used mostly for coaling purposes. Quite extensive reinforced concrete docks, constructed during the last year or two, are found at Newport (Mon.), Swancombe, Gravesend, Portencross, South Bank, Ipswich, Newlyn, and numerous other places.

A number of small reinforced concrete docks or pier-heads, other than those just mentioned, exist in England in connection with shipyards, etc. Such docks are found at Dumbarton Shipyard, on the Clyde, and at several similar establishments.

At first the Hennebique system prevailed in all English reinforced concrete dock construction, but, of late, several other systems have been introduced, the most pronounced of these being the Considère spirally armored concrete piling. Though different types of piles are used in English reinforced concrete dock work, there is a most thorough system of diagonal bracing with each type.

In using the Hennebique or other systems of dock construction, where lateral and diagonal braces of reinforced concrete are put in the structure, it appears that trouble has arisen, and might again arise, from the joints in the bracing system. As the foreign docks built on the Hennebique and similar systems seem to have been a success, it does not perhaps cause as much trouble as it did at first, or would appear to cause. In a concrete structure, of whatever design, built in the water, there is always the danger of cracks below the water line that cannot be seen and properly attended to.

In a recent address, Mr. Robert Porter, of *The London Times*, stated that "England is one big port". From the vast number of reinforced concrete docks at present in her harbors, it does not seem amiss to say that some day soon the ports of England will be, figuratively speaking, one big reinforced concrete dock, and it will be impossible to enter that country without passing over a structure of this type. In fact, current English technical publications plainly indicate that there

is hardly a port of any prominence along her coast where reinforced concrete construction is not now being carried on extensively, to the extent of five heavy coal tip docks in one harbor alone, due to the great economy of such structures over their old wooden predecessors, in the way of maintenance expenses.

In a recent rebuilding of the old wooden docks or wharves at Plymouth, constructed 30 years ago, the engineers of the Great Western Railroad have stated that concrete construction was adopted because "the cost would be about two-thirds of the cost of rebuilding in timber". These new concrete docks rest on Considère piles, and were designed for a live load of 400 lb. per sq. ft.

In closing this description of English reinforced concrete dock construction, it is well to take note of a few points in favor of this type, as set forth by an English engineer from his experience in that field:

1st. Easy to build;
2d. Indestructibility;
3d. Small cost of annual maintenance; and
4th. Easy to repair.

Spain.

In Spain a number of interesting reinforced concrete dock structures have been built. At Seville, on the Guadalquivir, 50 miles from the sea, are two reinforced concrete ore-loading wharves of some magnitude. At the outer end of each a series of well-based columns supports an ore-loading car-tipping gear. The ore trains run over an extensive concrete viaduct to this tipping gear, and the ore is dumped from the cars by mechanical means into vessels lying at the head of each dock.

A rather striking reinforced concrete dock structure was built several years ago at Aznal Collar mines, also on the Guadalquivir River. It is 532 ft. long and 17½ ft. wide. Each bent consists of two piles capped with a deep concrete beam. As the bent piles are only 10.8 ft. from center to center, the deck system is cantilevered out about 3¼ ft. beyond the supporting piles on each side. The dock carries an electric traveling crane, the track of which runs along the outer edge of the cantilevered decking. The crane has a capacity of 11 tons at a radius of 39½ ft.

France.

In discussing reinforced concrete dock construction as carried out in France, it is of interest to note that the first reinforced concrete pile was made in France by the noted French engineer, M. Hennebique, in 1896, since which time the reinforced concrete pile industry has grown to vast proportions, being used extensively by almost every civilized nation on the globe. From the successful manufacture and use of concrete piling has grown the construction of reinforced concrete docks. Though reinforced concrete construction seems to have been of a negative nature during the close of the 19th century, except in France, its development, as applied to docks, was due to the influence and perseverance of M. Hennebique, and that, too, in a country other than France; thus reiterating the old saying that "a prophet is not without honor, save in his own country". The first use of reinforced concrete sheet-piling was made at Sable d'Olonne, Bay of Biscay, France. Though these piles were constructed in 1898, as a reinforcement to an old masonry sea wall, they are said to be in as good condition as when first placed.

To record the most important reinforced concrete docks in France: A concrete pile and retaining wall dock was built in 1902 at Nantes, on the Loire, 40 miles from the Bay of Biscay, and in 1912 was reported to be in good condition. At Arcachon, on the Bay of Biscay, not far from Bordeaux, the French engineers have constructed a reinforced concrete recreation pier, several hundred feet long, with some attempts at the artistic, to harmonize with the surrounding landscape. In constructing a quay type of dock at Dives, near the mouth of the Seine, concrete sheet-piles of the Coignet type were used. They are 7½ ft. from center to center, with a concrete slab closing the gap between them. This type consists of a pile with a sort of two-wing arrangement. A cross-section of the complete pile resembles a very flat T-beam. The piles are of 10 by 12-in. section, 26½ ft. long. The wings are 5 in. thick, and of such width as to make the whole panel pile 4 ft. wide and 18 ft. deep, the pile part thus projecting about 8½ ft. below the panel part. This dock or quay is 190 ft. long. At Fecamp, 20 miles north of the mouth of the Seine, and exposed to the full sweep of the seas, there is a reinforced concrete dock, built in 1911 on the Hennebique system. The depth of water alongside the dock is 28 ft. at high tide. The dock is well braced by diagonal concrete struts, and

has a concrete deck-slab. Further to the eastward, near the Straits of Dover, at Boulogne, there is another reinforced concrete dock, built in 1906 in connection with a quay dock, the reinforced concrete section being 51 ft. wide and 1 050 ft. long.

As France is not a large maritime nation like England, the development of reinforced concrete docks in her ports has not been so extensive as in some of the other European countries.

Holland.

At Ymuiden a reinforced concrete landing stage was constructed in 1903. It consists of two rows of hollow cylinders of reinforced concrete, 8 ft. 2½ in. in diameter and 28 ft. 8 in. in height, filled with sand. The rows of cylinders are 21 ft. 4 in. apart athwart the deck, but 16 ft. 5 in. longitudinally. The deck consists of the usual concrete beam, slab system, and carries two railroad tracks, being designed to support a 45-ton locomotive. The working face of the dock has a wooden fender-pile protection. With the exception of an extension of this landing stage, no other reinforced concrete dock work standing in sea water has been undertaken in Holland; however, extensive reinforced concrete quay walls, etc., have been constructed during recent years at Rotterdam, a fresh-water harbor on the Rhine.

Italy.

Though rather extensive use has been made of mass and reinforced concrete in the development of Italian harbors, etc., the writer has records of only two reinforced concrete docks built in that country. One is at Pozzuoli, near Naples, a regular Hennebique type of pile construction, with a heavy shear leg derrick at its outer end, and the other at Ravenna, on the Adriatic Sea.

Germany.

The overseas shipping of Germany is confined almost entirely to the two ports, Hamburg and Bremerhaven, the former of which is 85 miles from the sea, so that, compared with England, the number of reinforced concrete pile docks standing in sea water in Germany is exceedingly small. Although the port of Hamburg is equipped with the most modern shipping facilities, most of the docks are of the heavy masonry quay type, and not the long piers so common in America. This

is due to the narrowness of the Elbe as compared with the broad expanse of water-front of other ports. Although the writer's file is lacking in records of reinforced concrete pile or similar docks as having been constructed in Germany, no doubt they exist and are of considerable magnitude. It is hoped that some of the members of the Society may be able to furnish full information concerning them.

As to reinforced concrete dock construction in Sweden, Norway, Denmark, and Russia, the records are extremely meager, perhaps because there are no docks of this type in these countries. Russian engineers, however, have built several concrete docks in the Baltic Sea, there being one at Touapse, a jetty built of reinforced caissons.

Although little, if any, information seems to be available covering the harbor work, etc., of the eastern part of the Mediterranean, in Grecian, Turkish, and Asia Minor ports, it would be plausible to consider that the concrete industry is well known in that part of the world, and that it has been used to considerable extent in eastern Mediterranean ports as well as in the Black Sea, in the construction of not only graving or dry docks, but also reinforced concrete docks and quays.

Africa.

At Alexandria, Egypt, there is a reinforced concrete lighthouse, in 30 ft. of water, exposed to violent storms and heavy seas. Its base is a truncated pyramid 45 ft. high. The slender main tower extends some 33 ft. above the base, its top being about 48 ft. above sea level. At one time this lighthouse was rammed by a 3 800-ton steamer, but it does not seem to have been injured, except that, since that time, it has leaned 4° from the vertical.

At Senegal, on the French West Coast, a reinforced concrete pile dock and jetty was built in 1912. The dock is 242 m. (794 ft.) long, of varying widths, covering an area of 2 500 sq. m. (26 900 sq. ft.). The jetty head is 60 by 10.3 m. (196.8 by 33.8 ft.), with an approach 140 m. (461 ft.) long and 2.65 m. (8.7 ft.) wide, giving the jetty a total area of 1 300 sq. m. (13 990 sq. ft.).

At Cape Town, a long reinforced concrete pier was finished recently. The concrete pile part is 800 ft. long and 45 ft. wide, except for the outer 75 ft., where it is 65 ft. wide. The piles are 14 in. square, 200 being used in the structure. They were driven in groups of four, 13 ft. and 10 ft. from center to center, with a longitudinal clearance of

27 ft. and an across-dock clearance of 18 ft. between each group of four, thus enabling small boats to pass under the dock as well as down the center aisle, as it were. The outer 75 ft. is well braced, the piles being close to one another.

Australia.

Australia and New Zealand are far distant, and thus perhaps few data reach the American engineer as to their developments in concrete. From what is known of several famous concrete structures in that part of the world, these countries are well in the fore as respects reinforced concrete. In fact, at Auckland, New Zealand, there are from four to six reinforced concrete docks, two of which, at least, are of extensive size, *viz.*, a railroad dock, 1 500 ft. long and 240 ft. wide, and Queen's Dock, 1 200 ft. long and 280 ft. wide.

It is of interest, in passing, to take note of a reinforced concrete lighthouse standing in the Straits of Malacca, a two-story closed-in structure, supported by seventeen reinforced concrete piles. On top of the main structure there is an open part consisting of reinforced concrete columns and braces supporting the light and its house.

India.

At Madras three reinforced concrete docks, of the bulkhead type, were constructed in 1911. They consist of a series of concrete piles driven uniformly 8 or 10 ft. apart, depending on the type of construction used. Back of these piles were placed reinforced concrete slabs, in the same way that wooden planks are placed back of and secured to wooden timbers where bulkheads are built to retain earth.

In one type, where the water is only 6 ft. deep and the dock has a freeboard of 10 ft., 18-in. piles, 10 ft. apart, with a penetration of 18 ft., were used. The thickness of the two lower concrete slabs is 12 in., and that of the top slab 15 in.

In another type, where the water is 9 ft. deep and the dock has 6 ft. of freeboard, 15-in. square piles, 8 ft. apart, were used, the concrete slabs being 6 in. thick. The tops of the piles are connected by reinforced jack-arches. In the third type, the piles are 15 in. square, 10 ft. apart, and are tied back to anchors 30 ft. behind the face of the dock. The water is only 4 ft. deep at the face of the dock. Each of these three walls is about 1 500 ft. long, their total length being 4 700 ft.

China.

What is said to have been the boldest undertaking in the construction of a reinforced concrete dock is found at Shanghai on the Whang Poo River, 50 miles from the open sea. The great difficulty encountered in the building of this structure and its accompanying warehouses was the excessive depth of the overlying river silt. The design of the structure itself presents nothing especially new, in comparison with other dock structures, as built in England. The fact that nothing but mud of various degrees of solidity, with layers of impure sand at irregular depths and a thin layer of gravel at a depth of 300 ft., covered the location of the proposed structure for a tested depth of 400 ft., gave a most unsatisfactory soil on which to construct a dock. At a depth of about 400 ft. there was a bed of gravel. The top layer of 25 ft., into which the concrete piles were driven, consisted of a solid crust of stiff sandy clay crossed with old creek beds filled with soft mud, rendering the surface somewhat irregular. The load on the piles had to be carried by the skin friction of this upper layer of supposedly firm ground. From a careful study of the situation, and from deduction obtained from old timber docks built in the same soil, a factor of 500 lb. per superficial foot of pile was fixed as the maximum carrying capacity of the piles, exclusive of their own weight.

The dock structure is 1 160 ft. long and 174 ft. wide, the rear 160 ft. being covered with two steel-frame sheds arranged in such a way as to leave an open space of about 250 ft. between them as an entrance way to the main office building. The decking of the dock was designed for a load of 500 lb. per sq. ft., with a deck load of 350 lb. per sq. ft. for the general structure. In general, the structure consists of a 5-in. deck-slab, deck-beam, and girder system of concrete construction, carried by 15 by 15-in. concrete piers, 15 ft. from center to center, each pier being supported by four 14-in. square concrete piles. At the face of the dock, where there is a large traveling crane, the columns are 18 in. square, and are carried by a group of six 14-in. square piles. The designed load per pier was 45 tons, or 11 tons per pile. The structure has the usual horizontal, cross, longitudinal, and diagonal bracing ties.

Due to the extraordinary conditions under which this dock was built, it is of interest to note a few facts as regards the pile-driving and the final loads carried. The piles were driven with a 3½-ton steam

hammer, the weight of the cylinder, which acted as a dead load on the pile during driving, being 1½ tons. The piston stroke was from 6 to 8 in. The piles had a penetration of about 25 ft., the final set being 1 in. under the last blow. It is stated by the engineer of this structure that "no anxiety was felt as to the future stability of the majority of the piles" under such an excessive set, as it was considered that their supporting power would be increased after the skin of water between the mud and pile had escaped and true skin friction was established. The resistance to driving increased very little after a 15-ft. penetration had been reached, and sometimes was less for a penetration of 25 ft. and above. On one occasion the piles sank so rapidly under the imposed dead weight of the hammer and cylinder that their descent had to be checked by holding up the dead weight of the hammer at the desired elevation. After 4 weeks, this group of four piles was tested with a load of 45 tons, with a 1¾-in. settlement in 2 weeks, when the settlement ceased. The piles were lengthened 8 ft., and then driven deeper. Though it took 80 blows to re-start the piles, they were finally driven with the same ease as at first. A subsequent test of 45 tons gave a settlement of 1½ in. No additional attempts were made to lengthen any of the piles. When completed, the dock was loaded to more than that for which it was designed (1 000 lb. per sq. ft., by mistake), with no sign of any settlement.

In general, the piles were from 6 to 8 weeks old before they were driven. The range of temperature was about 150°, which appeared to cause hair cracks in winter along the lines where work was stopped in the deck-slab. One of the leading dock engineers of England has remarked that this dock illustrates the great advantages of a concrete decking in its ability to distribute the superimposed loads and thereby assist the piles to carry a heavy burden. The larger part of this structure was moulded on land and put in place on the unit system, as it were, especially those parts below water, except the caps. In connection with this dock, there was also built a quay wall about 495 ft. long and 21 ft. wide, on one side of the property, for the berthing of lighters.

Japan.

The writer has no record of any reinforced concrete pile docks having been built in the development of Japanese harbors, yet he has no doubt that they exist. For 20 years, at least, the Japanese have

been using concrete in breakwaters to an enormous extent, and it is only a part of wisdom to consider that they have used it in other reinforced concrete structures in sea water. Recently, extensive developments have been made in the harbor of Kobe, where four concrete piers have been built, and of a most unique type of construction, *viz.*, reinforced concrete caissons built in the dry on a pile structure on the shores of the harbor, then lifted off this foundation by a specially designed depositing dock, carried into deep water, and put afloat by sinking the depositing dock and withdrawing it from beneath the concrete caissons. The caissons were then towed to and sunk in their proper places in the quay wall. Each caisson, 119 ft. long, from 35½ to 41½ ft. high, and from 24 to 30 ft. wide, has twenty open-top cells, and weighs from 1 900 to 2 400 tons.

Philippines.

It may not be exact to regard the Philippines as a foreign country, so far as concerns engineering work now being done, yet, in this paper, it has been classed as such, and note is here made of the reinforced concrete docks in those waters. The first dock of this type was built in Cavite at Manila in 1902, on what is known as the "Cushing System of Piers", *viz.*, cast-iron cylinders surrounding wooden piling and filled with concrete, the deck system consisting of steel girders and beams covered with a concrete slab. The dock is 408 ft. long, 75 ft. wide, and contains thirty-six cylinder piers 6 ft. in diameter. The structure was built by the United States Navy Department in connection with a coaling station.

Two docks of a similar design were built in 1906 by the Navy Department at Manila. One of these is 650 ft. long and 110 ft. wide; the other is 600 ft. long and 70 ft. wide. There are nearly 300 cylindrical piers in the two structures.

The Navy Department has also constructed two reinforced concrete docks at the Naval Station, Olongapo. One of these is 332 ft. long and 45 ft. wide; the other is 300 ft. long and 60 ft. wide. The piers or columns of both consist of hollow concrete cylinders, 2½ ft. in diameter, filled with concrete, and 12 ft. apart longitudinally and 18 ft. transversely. The decking consists of I-beams, encased in concrete, and these support a suitable concrete slab. There are light diagonal ties at each bent, running from the base of each outer column to the

top of each inshore column, but there does not seem to be any other lateral bracing.

Another reinforced concrete dock has been finished recently in the Philippines, at Iloilo, about 330 miles south of Manila. It runs along the face of an old sea wall, and consists of a series of transverse girders, 10 ft. from center to center, with a 12-in. concrete deck-slab. The outer end of each girder rests on a hollow cylindrical column, and the land ends are seated on a concrete pedestal built up from the natural slope of the bank. At the face of the wall, arches are inserted between the girders in order to provide additional strength to resist lateral stresses, as well as to give to the dock an artistic finish.

South America.

In view of the extensive harbor developments and the magnificent systems of docks, etc., in some of the ports of the leading South American countries, it is to be regretted that more general information concerning them is not readily available for the engineers of the United States. These engineering undertakings and port developments stand out as the most prominent of such world undertakings, especially at Buenos Aires, which city is conceded to have the finest shipping and port facilities in the world, with Montevideo a close second. It has been stated that almost all South American docks of any magnitude are being constructed of concrete, though the percentage of reinforced concrete structures among them is unknown to the writer.

In the foregoing the writer has attempted to outline the most important concrete docks, etc., constructed up to date in European and other countries, without any attempt to go into the technical side of their design, though some of them are more in the nature of semi-concrete and concrete steel docks than full concrete structures. The list is far from complete, but gives a general idea of the extensive use of reinforced concrete in dock construction in foreign ports.

Part II.—Reinforced Concrete Docks in the United States.

In recording what has been accomplished in the construction of reinforced concrete docks in North America, including Porto Rico, Cuba, the Canal Zone, and Canada, such undertakings are so few and of such recent date, compared with those in European countries,

that the art of building reinforced concrete docks in these countries may be said to be hardly beyond its infancy, especially as regards out-and-out docks or piers, as the American usually understands the word, *viz.*, long structures running out from the shore in such a way that vessels can lie on each side. Unfortunately, it will be necessary to make note of some failures among North American concrete docks.

Atlantic Coast.

Boston.—The first concrete dock built in Boston Harbor has perhaps caused more discussion as to the feasibility of using concrete in sea water than any other American structure of this type, and, therefore, is far-famed in itself. This dock, or pier, was built at the Charlestown Navy Yard about 14 years ago. The first section, consisting of a long, straight wall, was built in 1899-1900, without any resort to a coffer-dam. The other two sections, consisting of plain reinforced arches, 20 ft. wide, with spandrel walls, were constructed in 1901, the space between the two walls being filled with earth and stone. The first section was built of 1:2:3 concrete throughout, the concrete being all placed in the wet, with an open-top bucket. The second and third sections were of different mixtures, the main body being of 1:3:6 concrete, the outer 2 ft. consisting of a 1:2:4 mixture, and the whole exposed surface was faced with 3 in. of 1:1 mortar. In these two sections the concrete was placed very dry, as was the practice at that time.

During 1913 one of the Boston public docks, Pier No. 5, was partly rebuilt in concrete. This dock consisted of a timber platform deck, 50 ft. wide and 1 150 ft. long, on each side of a solid earth fill. The new wooden piles needed were driven, and all the piles, new and old, were cut off at mean high water and capped with a deep reinforced concrete beam running athwart the piles, in the way of curtain-walls. These curtain-walls support two longitudinal reinforced concrete beams, the third or inner beam resting on the earth retaining wall. There are also two additional concrete beams under the track along the outer face of the dock, with special supporting piles. On top of the curtain-wall and longitudinal girders is laid a reinforced concrete floor-slab with a 2-in. bitulithic top. Wooden longitudinal tie members are run from top of pile to top of pile under the concrete curtain-walls, the whole timber structure being well braced with piles and wooden ties. To provide against accident from disintegration

of the lower part of the curtain-walls, due to frost action or other cause, cast-iron columns, 30 in. long, are attached to the top of the piles and made integral with the curtain-walls, thus guarding against weakness in the concrete, rather than in the piles, as done in other harbors.

New York Harbor and New Jersey Coast.—A reinforced concrete dock of an experimental nature was constructed at Ellis Island, New York Harbor, in 1911. It is a rather small structure, 30 ft. wide and 50 ft. long, resting on thirty-six driven concrete piles 18 in. square. The piles were made of different mixtures, for experimental purposes, various kinds of water-proofing being used in order to determine their efficiency under the same conditions. This was the first complete concrete pile and deck dock built in New York Harbor.

During the past seven years a semi-concrete type of dock has been under development in New York Harbor, *viz.,* wooden piling, wooden caps, and concrete decking. In one dock, on the New Jersey side of the Hudson, the caps on top of the piles are also of concrete. In the final type, as worked out by the Department of Docks and Ferries,* the concrete slab rests directly on wooden caps secured to the tops of the wooden piles, a genuine flat slab between bents, the entire timber floor system being wholly eliminated. At present some 25 or 30 semi-concrete docks have been built on this system in New York Harbor. It has been stated authoritatively that they have proved a great success.

In building two semi-concrete docks at the Brooklyn Navy Yard a few years ago, the objectionable features of docks of the foregoing type—*viz.,* part of the wooden pilings and bracing exposed to wet and dry conditions between low water and the decking, and the wooden cap as an additional temporary item helping to support a permanent structure—were eliminated. In the two Navy Yard docks the wooden piles were cut off a little above low water and capped with a wooden grillage. Pre-moulded concrete columns, mixed with water-proofing compounds, were set on and dovetailed into the caps, and a concrete girder-beam and deck-slab system was worked over the tops of the columns. The wearing surface consists of a creosoted wooden block pavement. Down each side of the dock there is a standard-gauge railroad.

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* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 508.

A small concrete dock was constructed at Glen Cove as a yacht-landing, in the winter of 1909-10. It consists of eight reinforced concrete rock-filled caissons, supporting an overhead footbridge, the total length of the pier being about 330 ft.

Long Branch.—At Long Branch a Hennebique type of concrete pile dock was constructed in 1911, running some 848 ft. out into the Atlantic Ocean, as a boat landing and recreation pier. At present the pier is only 75 ft. wide, except for an 80-ft. length at its outer end, where its width is 150 ft., the intention being to make the whole pier of that width at some future time. The deck is 22 ft. above low water. The piles are 16 ft. from center to center longitudinally, but 20 ft. from center to center across the pier, except the outer two rows, which are 15 ft. from center to center. Most of the piles are of hollow cross-section, 22 in. external diameter, 13 in. internal diameter; the penetration was about 22 ft. To provide sufficient impermeability, the shells of the piles were made of 1 : 1½ : 3 concrete, the fill being of a weaker mixture. Apparently, no cross-bracing system was used, the outer end of the pier being stiffened laterally by inclined bracing piles at regular intervals.

Atlantic City.—At Atlantic City the famous steel pier was widened and protected in 1906 by the use of concrete. The original pier was founded on steel-pipe piles resting on cast-iron disks, a type of construction quite common during the last part of the 19th century. The pier extends out into the Atlantic Ocean a total length of 1 600 ft. In rebuilding the pier all the original metal work was encased in concrete and the pier was widened on both sides, 12-in. and 25-in. reinforced concrete piles, with enlarged footings, being used. The smaller piles were pre-moulded vertically, on a small platform, each pile at its final location. After hardening sufficiently they were lifted off their platforms and jetted into place through from 8 to 14 ft. of sand. The larger piles were given a penetration of 16 ft. The lower 12 ft. of these piles were pre-moulded on a platform, a water-tight iron casing was secured to the upper end, and the whole was jetted into place. The dry caisson was then filled with concrete up to the proper level, the maximum total length of the 25-in. piles being 52 ft. In protecting the original piles, concrete shells were cast around them, with sufficient interior clearance, and the space was afterward filled with grout.

After the concrete work of this structure had been in the sea water for 6 months, the piles became coated with a sort of gelatinous matter which seemed to act as a most excellent protective coating against any deterioration. The same peculiar action has also been noticed in California.

Although not exactly a dock, it is of interest to note the concrete pile Boardwalk at Atlantic City. Not only is it necessary to guard against dry and wet conditions at such resorts, but the fine sands act like a sand-blast when driven like snow before the wind. In 1908 part of the old wooden structure was rebuilt with 16-in. concrete piles, supporting a concrete cap, and that in turn carried the wooden decking.

Baltimore.—It is perhaps at Baltimore that the most extensive reinforced concrete docks on the Atlantic seaboard have been built. Although the water in Baltimore Harbor may not have the same density of salt as in ports nearer the sea, these docks, thus far, have shown no sign of deterioration, though at times subject to frost action. Three of these piers are of a back-filled concrete bulkhead type, and not docks resting on piles. Pier No. 4 is 978 ft. long and 220 ft. wide; Pier No. 5 is 1 245 ft. long and 200 ft. wide at the shore end, but 243 ft. wide at the water end; Pier No. 6 is 1 456 ft. long and 93 ft. wide at the shore end, but 212 ft. wide at the water end; all were built in 1908.

In general, these three docks consist of a series of oval-shaped concrete cylinders 25 ft. apart along the face of the docks, and sunk to about 25 ft. below low water. Along the face of the cylinders, and just above high water, there is a concrete-encased iron girder, tied back to a deadman some 28 ft. in the rear of each cylinder. A row of concrete sheet-piling was driven back of the girders to form a vertical retaining wall, the upper ends of the sheet-piling bearing against the girder and the lower ends being driven into the muddy bottom. A horizontal box-girder encased in concrete runs along the upper face of the dock, supporting the outer edge of the concrete curb slab, on which are laid the paving blocks. The cylinders are tied together in certain cases by ties extending entirely across the docks. The face of each dock is protected by wooden fender-piles, 8 ft. apart. Another concrete dock has been completed recently in Baltimore by the Harbor Commission, the details of which are lacking. In the

same harbor is found a concrete bulkhead dock, built for a private corporation—a reinforced concrete sheet-piling structure capped with a concrete girder tied back to deadmen by reinforced concrete ties.

At Sparrows Point, near Baltimore, a reinforced concrete ore dock, 600 ft. long, was built in 1911. It consists of two parallel concrete walls, about 46 ft. apart, *viz.*, (1) a sheet-pile bulkhead on the water-front capped by heavy concrete girders with a cantilevered shelf, as it were, on the outer face, running the full length of the bulkhead; (2) a heavy retaining wall in the rear, the two walls being tied together by reinforced concrete ties about 30 ft. apart. The back wall, resting on wooden piles, not only acts as a deadman for the outer wall, but affords a means for carrying one track of the large, heavy, ore unloading crane that straddles the filled-in space between the two walls, the front track of the crane running along the outer wall. The dock face is protected by a substantial system of fender-piles and wales with heavy helical car-springs at each buttress of the face wall.

Norfolk.—In constructing the Virginian Railway Coaling Terminal, at Sewells Point, in 1907, it was not practical to carry the massive steel superstructure on creosoted piles. In place thereof groups of wooden piles were driven and cut off 1 ft. below the mud line. On top of these piles were built monolithic concrete piers, of pyramidal shape, to 4 ft. above high water. All the concrete work was done in the dry, inside a coffer-dam. These piers are reported to be in as good condition as when first built.

Brunswick and Charleston.—Perhaps the most extensive development of concrete dock construction, combining concrete piles with wooden decking, is found at Brunswick, Ga., and at the U. S. Navy Yard, Charleston, S. C., built in 1906.

The Brunswick terminal consists of two piers, 500 and 900 ft. in length, respectively, and each is 140 ft. wide; there is also a coaling pier about 300 ft. long. The 16-in. bearing piles, of pre-moulded concrete, are 12 ft. from center to center each way. They are from 30 to 51 ft. in length, with the lower 10 ft. tapering to 8 in. The piles have a penetration of 40 ft. Their upper ends are corbeled out to support the double 8 by 16-in. wooden caps. The decking consists of 6 by 14-in. stringers and a 3-in. flooring. Each bent is well braced with creosoted wooden cross-bracing.

The Charleston Navy Yard dock is 60 ft. wide and 520 ft. long, and of the same type of construction as the Brunswick structures. The piles, 10 ft. from center to center each way, are 18 in. square, instead of 16 in. square, and have an 8-ft. taper to 12 in. square at their lower ends, thus giving a heavier structure than those at Brunswick. The test load on the Charleston dock was 30 tons per pile for 48 hours, though the specification required only 20 tons, or 400 lb. per sq. ft.

The outer row of piles in these docks consisted of three creosoted yellow pine sticks, two of which were driven on a batter; all were bolted together to afford sufficient protection to the dock in the form of a fender-pile system.

During the building of the Brunswick dock it was rammed by a large steamer. Although a number of the pine piles were broken, it has been stated that the concrete piles withstood the shock successfully.

Savannah.—A rather unique type of concrete dock was built at Savannah, 17 miles from the sea, in 1913. The design seems to contain many of the excellent features of the Ambursen dam. This dock consists of a series of pile bents athwart the dock supporting reinforced concrete brackets of triangular shape, the brackets in turn supporting a concrete deck-slab sloping down and toward the rear of the structure. This deck was afterward back-filled and finished off with a suitable working face. The dock forms a water-front structure, and is protected by wooden fender-piles.

Jacksonville.—In the construction of a semi-concrete dock at Jacksonville, the Braxten concrete pile was used, with very satisfactory results. In another case the Ripley concrete-encased wooden pile was adopted.

Key West.—A reinforced concrete quay wall dock, 1 589 ft. long, was completed at the U. S. Navy Yard at Key West in 1912. The main wall consists of a series of pre-moulded concrete pile bents capped by a concrete girder and a deck-slab 40 ft. wide on top. From the inner edge of the deck-slab a sloping concrete apron runs down to the top of a row of sheet-piles which forms a retaining wall for the reclaimed land. The piles are from 16½ to 20 in. square, and vary from 25 to 60 ft. in length. The bents are 10 ft. apart, with the same spacing for the piles, each bent having six piles. The face of the dock is protected by a system of creosoted fender-piles placed midway between each bent.

Port Arthur.—A reinforced concrete pile-bent dock, 1 050 ft. long and 25 ft. wide, was constructed at Port Arthur, Tex., in 1911-12. In general, the piles are 16 in. square, 44 ft. long, and 5½ ft. from center to center. The pile bents, of five piles each, are about 23 ft. apart, and are capped with a reinforced concrete girder. Five concrete beams, running from bent to bent, and a 4½-in. concrete slab, form the deck structure. The dock is tied back to the concrete trestle built for carrying the railroad tracks in the rear of the dock. No provision is made for any spring or other device to take up the impact forces on the fender system, as it is believed that the wooden fender-piles will afford sufficient elasticity to prevent any injury to the dock from this source.

Cuba.—Two reinforced concrete docks, 620 and 670 ft. in length, respectively, and 160 ft. wide, were built in 1911-12 at Havana, the depth of the water varying from 12 to 40 ft. Each consists of a concrete floor-slab resting directly on concrete caps placed on top of clusters of from four to eighteen reinforced concrete piles, the clusters being about 23 ft. from center to center in each direction. The concrete piles, 18 and 20 in. square, were designed for a load of 32 tons each. The design of the floor slab would indicate a cantilever effect longitudinally between each row of longitudinal piling.

One of the railroad companies of Cuba, also, has built a reinforced concrete dock at Havana, for coaling purposes. The structure, which is subjected to very heavy loading, rests on Chenoweth concrete piling, and was but recently finished.

Haiti.—In constructing a reinforced concrete dock at Port au Prince, during 1913, the Ripley type of concrete wrapped wooden pile was adopted. This dock has a total length of 2 326 ft., varying in width from 24 to 60 ft. The piles are 10 ft. from center to center, longitudinally and transversely, and are capped by heavy concrete girders of rectangular section for the inshore end of the dock, otherwise by arched girders. The deck system consists of a series of reinforced concrete beams supporting the concrete deck-slab, built with a crown, in order to shed water. The dock is protected by a creosoted fender-pile system.

Panama and Canal Zone.—The United Fruit Company in 1909 built a combined reinforced concrete and wooden pile dock at Bocas del Toro, for the docking of fairly large steamers. The wooden piling is surrounded by a 4-in. concrete shell up to about 1 ft. above the

high-water line. The piles are extended up to the deck as reinforced columns, with a concrete beam and deck-slab system. Up to the present time the dock is said to have given good results.

In the Canal Zone the U. S. Army Engineers have constructed a reinforced concrete dock, 706 ft. long and 55 ft. wide, for unloading timber. There are fifty-five concrete piers or columns, 8 ft. in diameter and about 80 ft. long arranged in two rows, 35 ft. from center to center across the dock, and 30 ft. from center to center longitudinally, and built in the form of hollow reinforced concrete sectional cylinders. After these cylinders had been sunk to bed-rock, they were filled with concrete, being reinforced vertically with eight rails. On top of the columns there is a concrete girder, deck-beam, slab system. The girders are about $5\frac{1}{2}$ ft. deep, the beams about $4\frac{1}{2}$ ft. deep, and the slab 6 in. thick. The railroad track runs over one row of columns. The floor system is designed for a load of 400 lb. per sq. ft., with a concentrated load of 105 tons over the track beams. The depth of water for a mean sea-level tide is 40 ft., the total fluctuation in the tide being 20 ft.

Two other concrete docks of extensive size are now in course of construction in the Canal Zone, with still more to follow.

Pacific Coast.

On the long stretch of our Pacific Coast, perhaps is found the greatest development of reinforced concrete dock construction in the United States. This section is making vast harbor improvements in anticipation of the opening of the Panama Canal.

San Diego.—At this most southern port on the California Coast an extensive reinforced concrete dock is now under construction. It consists of two parts, *viz.,* the dock itself, 800 ft. long and 130 ft. wide, and a quay wall or bulkhead, 2 675 ft. long and 25 ft. wide. Wooden piles are driven into the soil and cut off "at any point between mean low water and 18 ft. below city datum." Each of the 42-in. concrete columns encases one wooden pile and supports a system of structural deck-beams, a concrete slab covering the whole. The columns are 15 and 13 ft. 4 in. from center to center. The entire structure is protected by a wooden fender-pile system having the so-called San Francisco type of steel spring shock-absorbers.

San Pedro.—In connection with extensive port developments at San Pedro, a semi-reinforced concrete dock was recently completed in the outer harbor. It consists of pre-moulded concrete piles, 10 ft. from center to center in each bent, the bents being 16 ft. apart. The tops of these piles are corbeled out to support two 10 by 16-in. wooden caps, which in turn support the wooden floor joist and wooden decking. The piles are tied together with a wooden cross-bracing system above mean high tide. The structure is also stiffened against lateral blows on its face by inclined bracing piles. The wooden pile fender system has a car-spring to assist in taking up lateral forces. The dock is of the quay type, 48 ft. 6 in. in width, the total pier head frontage being 12 000 lin. ft. A railroad runs parallel to the inner edge on the inshore fill.

Redondo.—It is of interest to take note of the ocean pier at Redondo, Cal., though it is not a dock. It extends some 637 ft. out into the Pacific Ocean, and supports the intake pipe (for cooling purposes) of a power station. The pier consists of concrete pile bents, 20 ft. apart, each bent having four piles. As considerable surf runs at times under this pier, the outer bents have an extra outside pile driven with a batter of 2 in. per ft. The piles consist of a thin steel shell, 18 in. in diameter, closed at the lower end. After the steel cylinder had been driven to the proper depth of penetration, the reinforcement was inserted and the cylinder was filled with concrete. The piles of each bent have a structural steel cap encased in concrete, with a system of diagonal bracing in a horizontal plane connecting the tops; there is also a longitudinal system of ties at the tops of the piles, above the reach of the water. This structure, as a whole, is said to possess considerable elasticity.

Long Beach, Cal.—In 1907 a concrete pile pier was built at Long Beach, extending some 1 300 ft. out into the ocean. The head of the pier is 100 ft. long and 300 ft. wide, the approach being 1 299 ft. long and 32 ft. wide. The deck is 30 ft. above mean low water, so as to be kept clear of the 24-ft. waves which at times roll in from the ocean. The piles and columns are 4½ ft. in diameter, and are arranged in bents. Under the head of the pier the bents and piles are 16 ft. from center to center. The approach bents are 20 ft. apart, with two columns each. The columns are sunk from 10 to 18 ft. into the sand, in order to be absolutely safe, as it is said that at times the undertow

digs out the sand to a depth of 13 ft. The pile caps consist of steel I-beams, on top of which rest wooden stringers carrying a wooden decking.

Santa Monica.—At Santa Monica, an ocean pier, built in 1908-09, extends 1 600 ft. out into the ocean. It was built on driven concrete piling, ranging from 14 to 22 in. in diameter, in lengths up to 75 ft., with from 16 to 20 ft. penetration. Although the pier was built primarily to support the outfall sewer carrying the sewage effluent to a point far seaward, it is also used for recreation purposes. The pier is about 35 ft. wide at the deck line, with three platform spaces of 43 by 89 ft. at intermediate points and at the end. The bents are 20 ft. from center to center and consist of three piles, the piles being 13 ft. 6 in. from center to center. Each bent has a concrete cap on which rest the wooden joists covered with 2-in. planking, and the latter is covered with a 3-in. wire-mesh concrete slab, having the proper pitch to carry off the water. The bents are tied together by three longitudinal reinforced concrete tie-beams running from top of pile to top of pile. The piles are bulb-pointed.

San Francisco Bay.—Although the dock engineers of New York City have developed a type of semi-reinforced concrete dock, *viz.*, a wooden pile structure supporting a concrete slab, especially adaptable to local conditions, the dock engineers of San Francisco have developed a type of full reinforced concrete dock based on wooden sub-piling, concrete column piers, steel or concrete deck-beams, and concrete floor slab, the concrete encasing the steel beams and the floor being made monolithic, with details varying to suit special conditions. Although the type as worked out presents no difficulty in the way of construction, outside of building the main columns, that part of the work has been done successfully, but with considerable difficulty. The mud line at the bottom of the bay is said to be approximately level, yet, at the outer ends of some of the piers there is a depth of only about 18 in. of mud over the rock; at the shore end, however, there is a depth of 35 ft. of mud. Piles can be driven to a rock bearing in some places, but it is impossible to use wooden piles throughout. Along a portion of the water-front, where it is not possible to reach the rock, there is a hard soil capable of bearing from 4 to 6 tons per sq. ft., thus doing away with the necessity of any sub-piling.

The method used in building the column piers is to sink a hollow

steel caisson, of such length that it will not be overtopped by the water, dredge out the interior to the desired depth, and build the reinforced column in the dry. In some cases the columns rest on solid rock, in others, wooden piles have to be driven inside the cylinders to obtain the necessary bearing support. The size of the columns varies according to conditions. In two docks built in 1910, 140 ft. wide and 780 ft. long, where the mud covering the rock was less than 50 ft., the columns were seated directly on the rock. Where the mud is more than 50 ft. deep the columns rest on five 15-in. wooden piles driven to refusal, the piles being cut off 35 ft. below the water line and encased by the concrete columns to that height. The columns are 6 ft. in diameter to a height of 7 ft., and then 3½ ft. to the top.

In laying out a vast dock improvement proposition at Fort Mason, San Francisco, the Government has planned for the immediate construction of three docks of the usual San Francisco type, each to be 500 ft. long, two 81 ft. wide, the third 118 ft. wide. The concrete columns are to be supported by groups of seven wooden piles driven in a circle 6½ ft. in diameter, and 18½ ft. from center to center each way. The piers are to be 8 ft. in diameter up to 12 ft. above the dredge line, and are then to be reduced to a diameter of 4 ft. for the remainder of their length. The wooden piles will extend some 11 ft. up into the concrete columns, the bottom of the concrete being well below the mud line. In building the first of these docks, an attempt was made to construct the column forms of 4-in. staves, sufficiently reinforced with bands, and sink them into position by driving. The method did not prove a success, and resort was made to the steel cylinder caisson method, as described previously.

Up to 1911 there were only four modern reinforced concrete column docks under the control of the San Francisco port authorities. Since that time they have added largely to the number by replacing some of the older wooden pile docks with reinforced concrete structures. The first addition was Pier No. 17, 800 ft. long and 126 ft. wide, with suitable railroad track accommodations. It consists of wooden piles protected by concrete shells, the deck-beams being of structural steel encased in concrete, and the stringers and decking are of timber—a sort of semi-concrete pile semi-concrete dock.

The next docks reconstructed were Piers Nos. 26, 28, 30, and 32, all of the same type, having reinforced concrete columns resting on

the hard bottom, without any piling, with a complete system of reinforced concrete deck-beams, girders, and slabs. These docks are equipped with up-to-date cargo-handling machinery.

The next addition was Pier No. 39, 150 ft. wide, this being in process of construction at the present time. The concrete columns rest on groups of from 4 to 10 wooden piles, the entire deck system being of reinforced concrete.

In another type of construction at San Francisco the wooden piles are wrapped with wire fabric, or otherwise, and a concrete shell is placed around them after the piles have been driven to place. This method, apparently, has proved successful, though it must be carried on in such a way that the concrete can be poured, set, and hardened in the dry, and not in sea water, if permanent results are to be obtained.

Recently, the City of Oakland, Cal., built a genuine reinforced concrete pile dock, 295 ft. long and 124 ft. wide, standing in 30 ft. of water. The piles are of pre-moulded concrete, 16 in. in diameter, octagonal, and of a 1 : 1½ : 3 mixture for a distance of 5 ft. from the top, the remainder of the pile being of a 1 : 2 : 4 mixture. The bents and piles are approximately 10 ft. apart. Each row of piles has a concrete cap or girder running athwart the dock, the deck-beams and deck-slab being also of concrete. For lateral stiffness, 12-in. concrete curtain-walls were built at three points in the dock, for about one-third of its width, between the piles in three bents.

Portland.—Being 112 miles from the ocean, on a fresh-water river, it is possible to use wooden piling at Portland, in the construction of docks. A massive concrete dock terminal, now being built by the city, consists of four concrete warehouses along the water-front. The dock part of this project as designed consists of a reinforced concrete platform, 1 030 ft. long and 100 ft. wide, 32 ft. above low water, resting on wooden piles driven in groups and cut off at about mean low water. Resting on each group of wooden piles, 20 ft. from center to center, are the reinforced columns supporting the upper platform, composed of steel I-beams encased in concrete, and a concrete floor-slab. For a length of 300 ft., a low-level deck, 14 ft. below the main deck, is provided. As the Columbia River is subjected to high- and low-water stages, due to floods, it was necessary to provide this lower platform for use by river steamers during low-water periods. The rise

and fall of the Columbia River attains a maximum of about 28 ft., though 18 ft. is about the average, all based on mean low-water level. Thus the lower platform will seldom be under water.

Puget Sound.—Though there are several large shipping ports on Puget Sound, up to the present time, no reinforced concrete dock construction of a commercial nature has been undertaken in these waters. As lumber is so plentiful in that part of the country, it is only natural that such a section should be one of the last shipping centers to take up the building of reinforced concrete docks; but as the destructive teredo is very active there, the engineers of the Northwest are beginning to seek a more stable type of construction than creosoted wooden pile docks, especially the United States and Canadian Governments, and some of the railroads, because "in Government [and publicly owned] docks, a small saving in first cost is of minor importance, but weakness and frequent need of repairs are well nigh intolerable. On the other hand, in private ownership of docks a saving in first cost is usually of serious importance, while the cost of maintenance, repairs, etc., is met by earnings of the dock and is less felt", unless they become so excessive as to make a concrete proposition more economical in the long run.

At the U. S. Navy Yard in Bremerton, Wash., a reinforced concrete dock, consisting of concrete columns, steel and concrete beams, and a concrete slab, was completed in 1912. It is 402 ft. long and 60 ft. wide. The columns are 3 ft. in diameter, with a flared-out footing to a diameter of 6 ft., 16 ft. from center to center, each way, there being four columns to each bent. The caps and girders over the tops of the columns are of I-beams encased in concrete. A concrete beam is run midway longitudinally between each of the four steel beams. The I-beam caps are cantilevered out 6 ft. on each side of the dock. The columns were built as hollow concrete cylinders with a 3-in. shell of 1:2 mortar, and filled with a 1:2:4 concrete after being sunk to place. As hollow cylinders, they avoided any coffer-dam work. The dock has a standard railroad track down each side over the outside columns, also an ordinary wooden fender system.

A still heavier concrete dock is now in course of construction at the same Navy Yard. It is 490 ft. long, 80 ft. wide, and is designed for a load of 600 lb. per sq. ft. The approach of the dock consists of a wooden pile structure, 210 ft. long, of triangular shape. As designed, the

structure is supported by sixty-eight concrete columns, 4 ft. in diameter, with an 11-ft. base (of the same type as in the dock just mentioned), 20 ft. from center to center athwart the dock, and 30 ft. longitudinally. The columns are capped by extremely heavy reinforced concrete beams which support a series of built-up structural beams, about 36 in. deep, carrying a thick concrete floor-slab. The side of the dock is cantilevered out 8 ft. beyond the columns. A standard railroad track runs over the center line of the outside rows of columns on each side of the dock. On account of the nature of the soil, some of the piers rest on sub-piling, others, nearer the shore, on hardpan.*

Vancouver.—The Great Northern Railroad has very recently completed an extensive reinforced concrete dock of the quay type, in connection with a new terminal it is building at Vancouver. The total width of the terminal dock is 302 ft., the concrete dock proper being 456 ft. long, and 50 ft. wide on each side of the terminal, the space between being a rock and earth fill, with a proper rip-rapped slope at its faces. Each concrete dock structure consists of nineteen reinforced concrete columns, 4½ ft. in diameter, 25 ft. from center to center, parallel to the face of the dock, resting on the rock stratum that underlies the location of the terminal. These columns support a heavy longitudinal concrete girder into which are tied heavy cross-girders, 12½ ft. apart. The cross-girders are cantilevered out beyond the longitudinal girder about 16½ ft., and their inboard ends rest on two driven concrete piles 34 ft. back from the center of the columns. Four concrete beams of suitable size are run longitudinally with the dock. The entire girder and beam structure supports a concrete slab. The railroad track is over the longitudinal girder running from column to column. The two parts of the dock are tied together by suitable concrete beams running across the interior fill. The terminal is considered to be one of the most substantial and up-to-date structures on the Pacific Coast. The dock is well protected by a wooden fender-pile steel-spring system.

Great Lakes.

Although extensive use of concrete has been made in some of the ports of Lakes Superior, Michigan, Huron, Erie, and Ontario, in the construction of massive ore-docks, it is not proposed to discuss that particular phase of the subject in this paper. These docks are of

* In the actual construction the built-up structural beams were replaced by reinforced concrete.

excellent design, some built on concrete piling, others on wooden piling or cribs. Concrete piles in fresh water are not subjected to the same deterioration as those in salt water. On the other hand, they have to withstand extremes of temperature, frost and ice, in addition to severe treatment, due to the heavy traveling machinery above them.

Some of these docks will be discussed, in order to bring out the important features of their special design.

Chicago.—One of the first attempts at using concrete for dock work on the Great Lakes was made at South Chicago, in the winter of 1898-99, in the rebuilding of an old wooden quay dock wall. The concrete structure is 1 680 ft. long, and consists of a heavy mass concrete stepped-back wall, 10½ ft. high, 8 ft. wide on top, and 18 ft. on the bottom, supported by a timber structure consisting of three rows of piling cut off 3½ ft. below mean water level, with longitudinal caps crossed by a heavy wooden grillage. A timber sheet-piling bulkhead under the face of the wall acts as a retainer for the slag fill. On one occasion the wall was rammed by a large steamer, but suffered absolutely no damage. The damage to the steamer, however, was rather extensive. A year or so later, a similar quay dock wall, 2 300 ft. long, was constructed at the same steel plant.

Due to the decay of a long wooden water-front bulkhead on the Chicago River, it became necessary to replace it. This was done by constructing a reinforced sheet-pile bulkhead almost ½ mile in length, capped by an I-shaped concrete beam, some 3 ft. wide and 5 ft. high. This beam rests on pre-moulded piles, which are 20 ft. apart and are secured to buttresses, also 20 ft. apart, which run back about 12 ft. from the sheet-piling, the land ends of the buttresses being supported by other piles. The buttresses are also tied back to deadmen some 35 ft. back from the wall. As the bulkhead has about 18 ft. of water along its front, vessels can dock alongside.

Marquette.—The most extensive reinforced concrete ore-loading docks on the Great Lakes are without doubt at Marquette, Mich., and were built in 1912. The substructure consists of a heavy concrete slab and facing-walls—similar to a channel beam placed on its back—1 500 ft. long and 60 ft. wide, resting on wooden piling. Under the face of the walls there is a wooden sheet-piling bulkhead which retains the sand fill around the piles under the dock structure. The depth of

the concrete web is 3 ft., the walls that correspond to the flanges being 9 ft. high. The substructure supports a very massive reinforced concrete superstructure for loading ore into vessels, the whole dock being a very substantial and shock-resisting structure. A steel plate is worked along the face of the two concrete walls from 6 in. below the water level to 3 ft. above it, to prevent disintegration of the concrete due to frost and ice action at the water level.

Two Harbors.—At Two Harbors there is an ore-loading dock consisting of a steel superstructure supported by a mass concrete wall running along each face of the dock, and tied together with concrete beams at regular intervals. The concrete walls rest on wooden piles with a wooden sheet-piling bulkhead to retain the interior fill. The substructure is 1 400 ft. long and 52 ft. wide.

Detroit.—The concrete ore-loading dock at Detroit is 200 ft. long. It consists of three bearing walls of a T-rail shape, 9 ft. deep, running parallel to each other, the two outer walls being 28 ft. apart, the third or back wall being 173 ft. from the middle wall. The three walls rest on a double row of oak piling cut off 3 in. above the water level. Being capped by the concrete walls, no part of the wooden piling is exposed to the air. The outer wall stands in about 10 ft. of water, the middle wall in 3 ft., and the rear wall is far back on the dry land. The two outer walls are tied together by reinforced concrete beams, 5 ft. deep, at intervals of 10 ft., with cantilevered brackets on the water face of the outer wall opposite the tie-beams. The concrete deck-slab on top of the brackets is 12 in. thick, but only 6 in. thick between the two outer walls. The dock has a suitable wooden fender-pile system.

Between the middle and back walls there is a 12-in. reinforced concrete floor-slab, supported on a series of piles, 5 ft. from center to center each way, stated to have been designed for a load of 6 800 lb. per sq. ft., or 85 tons per pile. The ore floor is tied into the middle and back bearing walls opposite each cross-beam in the dock proper. The three walls support an ore bridge tower, used for unloading ore from vessels to cars on the track just behind the middle wall, or into the ore bin between the middle and back walls.

Toledo.—At Toledo, Ohio, there is an ore-unloading dock, consisting of a plain concrete wall running along the water-front, resting on wooden piles, and tied back to deadmen some 100 ft. in the rear. The

land is reclaimed back of the middle wall, a row of sheet-piling under the river wall retaining the earth in place. Parallel to the river there are three other walls to support the legs of the ore-unloading bridges which run on tracks placed on all four walls, spanning the four car tracks and the ore piles. The distance from the front wall to the inmost land wall is 419 ft.

Cleveland.—There is a full reinforced concrete ore-unloading dock at Cleveland, Ohio, completed in 1912 by the Pennsylvania Railroad. It consists of a reinforced concrete water-front wall, 985 ft. long, supported by a double row of pre-moulded concrete piles. The face wall is tied back by reinforced concrete tie-beams to another reinforced concrete wall supported by three rows of concrete piles about 81½ ft. in the rear. The tie-beams are 30 ft. apart, and rest on concrete piles 6 ft. apart. The space between the two walls, that is, under the concrete tie-beams, is back-filled with rip-rap, the whole mass being pumped full of sand. A concrete sheet-piling bulkhead under the front wall retains the fill in place. The dock has the usual wooden fender-pile protection. The tracks for four 17-ton Hewitt unloading machines run along the inner and outer walls. So far as the writer has been able to discover, this was the first concrete pile dock built on the Great Lakes.

Another ore-unloading dock at Cleveland consists of a reinforced wall of the same type as that at Detroit, the two structures having been designed by the same engineers. The cantilevered face of this wall is supported by brackets at intervals of 15 ft., with rear buttresses opposite them. The buttresses rest on a concrete base slab, the rear edge of which rests on a row of wooden sheet-piling, the base slab evidently being monolithic with the rest of the wall. The space between the sheet-piling and the wooden piles that support the concrete wall (11 ft.) is filled with slag. In general, the cross-section of the wall is dumb-bell shaped.

General.—In discussing the ore docks of the Great Lakes, it is necessary to consider the special and peculiar conditions under which they are operated, as they are designed to meet these conditions, and not *vice versa.* The larger portion of the freight carried on the Great Lakes is of heavy bulk form, *viz.,* ore and grain on the downward trip, coal on the upward trip. Thus ore and coal cargoes form the two heaviest items handled in bulk masses at the terminals. Nowhere in

the world is found such massive and modern machinery for the economical and expeditious handling of bulk freight as at the upper and lower ports of the Great Lakes. Consequently, such docks, wooden or otherwise, must not be judged by the type used along the seaboard of the Atlantic, the Gulf, or the Pacific.

It appears that the various types of reinforced concrete docks as worked out by the American engineers are far more numerous than in foreign practice, as is evident from the foregoing descriptions. Although the Atlantic Coast engineers seem to favor premoulded concrete piles, the Pacific Coast engineers apparently favor large concrete columns. Perhaps in time a typical American concrete dock will be designed or devised, as in the case of the long-standing type of wooden pile dock structures.

In the foregoing review of American reinforced concrete docks, an effort has been made to include each and every port wherein such types of docks exist, as well as to mention each and every dock already built, so far as the writer has been able to acquire sufficient information concerning them, in order that the exact situation as regards the development of reinforced concrete dock construction in America up to the present time may be known to all. If the writer has unintentionally omitted any such dock structure, he will be pleased to receive information relating to it.

The writer has endeavored to determine which was the first complete reinforced concrete dock constructed in the United States, but has been unable to do so. The concrete dock wall built at Chicago in 1898-99 appears to have been the first of its kind constructed on the Great Lakes. A study of the constructive dates of concrete pile or concrete column docks would indicate that such types began about 1905 or 1906. Still, it is not evident which was the first of such docks to come into existence, the whole development being a gradual evolution from a concrete-filled steel cylinder column, steel deck-beams, and concrete-slab type, as used in the Philippines by the United States Government in 1902.

Irrespective of the actual beginning of constructing reinforced concrete docks, it is generally conceded that Oscar F. Lackey, M. Am. Soc. C. E., Harbor Engineer of Baltimore, was among the first, if not the first, to blaze the way for the extensive use of reinforced concrete in dock construction in United States harbors.

Part III.—Failures of Reinforced Concrete Structures Subjected to Salt Water Action.

In discussing reinforced concrete docks, the fact that there have been failures among them must not be overlooked. In Massachusetts waters, north of Cape Cod, a number of serious cases of deterioration of concrete have been caused by the disintegrating effects of sea water, wave action, and frost, especially in Boston Harbor, where nearly all the concrete structures standing in sea water have been affected badly between high and low tide, the most notable instance of which is the concrete pier at the Charlestown Navy Yard. Although that part of the pier which is constantly submerged has given but little trouble, the part exposed alternately to the sea and air has been seriously affected, many large pieces having broken completely away, making it self-evident that some other agent than the chemical action between cement and sea water was at work.

As is well known, winter temperatures on the whole eastern front of the New England Coast run far below zero. In Boston Harbor 12° below zero is not uncommon. In the same way that hard earth and porous rocks are broken up by frost action, permeable concrete in freezing water will gradually be destroyed between wind and sea, as the water which gets into the concrete simply exercises its natural expanding function in freezing, which *a priori* is detrimental to the concrete structure. It is generally admitted that the exterior concrete in these Boston structures, especially in the Navy Yard pier, has failed almost entirely from the effect of the alternate freezing and thawing with each tide during the winter, due to permeable concrete.

A number of failures similar to that already cited have occurred in Boston, the disintegration taking place in all cases between low and high tides. In the case of the Dover Street draw-bridge pier, built in 1894, the disintegration had extended 1.4 ft. into the pier at the end of 17 years, the greatest damage being just below high-tide level. The pier was built of 1:2:5 concrete, with a 1-in. plastered mortar facing. English Portland cement was used throughout. Whether the 1-in. facing mortar was expected to act as a water-proof shield to the interior concrete is not apparent. Evidently, it did not act thus, as might have been expected.

As all the concrete in these disastrous cases seems to have been placed in the wet, that is, the sea water was allowed to come in con-

tact with the concrete before it had become thoroughly cured and hardened, such results are not to be wondered at, for one of the axioms of a successful use of concrete in sea water is that it must be kept from contact with sea water for such a period of time as to enable it to become thoroughly hardened, especially that part between tides in freezing climates.

In several cases in Boston Harbor where the concrete was placed inside of a coffer-dam, or used in the form of pre-moulded, driven, concrete piles, the concrete does not seem to have been affected as in the other cases cited. These successful cases go a long way toward substantiating the truism that concrete, to be used successfully in sea water, especially in freezing water, must be made impermeable in the process of making, with full consideration given to the brand of cement used, the mixture, the sand, and stone (or gravel), the skilled labor of placing, as well as keeping it from contact with sea water until it has set and hardened sufficiently. It is very apparent, from a study of the method used in placing the concrete in the disintegrated structures in Boston Harbor, that that method was far from possessing the essential features necessary for a successful solution of the problem, viewed in the light of present-day knowledge.

In comparison with these Boston failures, it is fitting to state that at Dundee, Scotland, where the climatic conditions are said to be worse than at Boston, and where there is a rise and fall of the tide of about 12 ft., the combined action of the sea, waves, and frost has had no ill effect on the concrete docks in that harbor, the concrete piles of which were allowed to harden for 30 days before being put in place.

Another noted case of the destruction of concrete by frost and sea action is the large concrete sea wall along the water front of Lynn, Mass.—a massive concrete sea wall exposed to the pounding of the winter storms and seas. The steps to the beach in the front of this wall were destroyed to such an extent as to be hardly recognizable as steps. It might be of interest to state that this wall and some of the damaged structures in Boston Harbor have apparently been repaired effectively by the cement gun process.

In reviewing these failures in Boston and vicinity, it is well to consider the results obtained in using concrete in another port subject to freezing and ice conditions, *viz.,* New York Harbor. In addition to freezing conditions, New York Harbor has to contend with a

strong tidal effect, which results in large solid ice floes and fields of broken ice moving back and forth with a tide of considerable velocity, ice floes of such size coming down the Hudson as at times practically to compel abandonment of all transfer traffic in that river. This is an effect from which Boston docks are perhaps free, as no large rivers flow into that harbor, the Charles being kept under control by the Charles River Dam.

In discussing this additional handicap and destructive force at work on New York City's 8¼ miles of concrete sea walls, some of which have been in existence for 41 years, Charles W. Staniford, M. Am. Soc. C. E., Chief Engineer of the New York Department of Docks and Ferries, states:

"Up to the present time [August, 1911], no disintegration has been discovered that can be attributed to the existence of the structure in salt water. The concrete itself is in an admirable state of preservation, absolutely hard, and is undergoing no regular process of disintegration." * * * "this sea-wall which has been under construction * * * for 41 years, is at the present time an excellent piece of work and is subject to the same climatic conditions as all cities on the Northern Atlantic Coast with the attending ice, cold and rain characteristic of this latitude."

In many instances, parts of this wall above low water are faced with granite blocks. This is a noted example of what can be expected in the way of using mass concrete in sea water if properly made, though perhaps some repair work has been necessary in order to maintain the excellent condition of the wall.

In some of the earlier sections of this wall the concrete was placed *"en masse, in situ"*, but, since 1876, most of the wall has been built by the concrete block method. Only under specially favorable conditions is it possible to place concrete successfully *in situ* under (sea) water, as it becomes disintegrated "through the chemical action of the sulphate of magnesia on fresh concrete or through the resulting porosity of concrete due to the impossibility of tamping under water";* the viscosity and weight of the mass not being sufficient to produce such a dense material as obtained in block work.

To discuss an opposite case in New York Harbor, *viz.*, Dry Dock No. 2, New York Navy Yard, originally built of timber in 1890, the

* This subject is discussed further by the writer in an article entitled "Chemistry of Salt Water Cement," *Metallurgical and Chemical Engineering*, January and February, 1914.

history of which it is not necessary to relate here: In 1900 this dock was rebuilt, concrete being used very extensively. During 1913 a large sum was expended in repairing and replacing the concrete altars and floors. As it has been stated that the difficulties of using concrete in sea water have been so great at this yard as to indicate that this is not a permanent material for use in sea water structures, it would be of deep interest to learn the facts as to the chemical composition of the cement used, of the sand and stone, as to the mixture thereof, and the precautions taken in mixing and placing, also as to whether the dock is kept flooded when not in use, especially during the winter. If, as has been stated, the concrete "has deteriorated and disintegrated to such an extent that it was possible to use a pick and shovel in removing it", it is apparent that it was lacking in one or more of the essential features that are deemed absolutely necessary for a successful use of concrete in sea water structures.

Whether any of the concrete pile docks on the Great Lakes have shown any signs of deterioration due to frost action, the writer does not know, but trusts that some facts covering this question will be brought out in the discussion. As the water level is practically the same all through the winter, only a very short length of the pile would be affected, and not some 10 ft., as in Boston Harbor.

One of the first concrete docks built in San Francisco is said to have failed in part, due to poor construction. The early method of building the concrete columns of San Francisco concrete docks was to use a wooden cylinder, strongly built, as a column form, into which, it has been stated, the concrete was poured, apparently without any attempt to pump out the cylinder. As long as the wooden cylindrical forms remained in place around the supposedly concrete column, the dock was pronounced a success. When the teredo had finally destroyed the forms, the columns began to collapse and the dock became a pronounced failure, because, in pouring the concrete, the heavier material —the stone or gravel—settled first, then the sand, and finally the cement. The result was that throughout the length of the concrete columns there were alternate layers of uncemented stone and sand, with the cement in between the sand of one batch and the stone of the following one. Concrete can be and is successfully dropped through a height of 50 ft.—and even up to 1 000 ft. in one noted case

in Arizona—but, if the receptacle into which it is dropped is full of water, disaster alone awaits the unfortunate engineer.

In another of the San Francisco docks, where wooden piles supported the concrete columns, the concrete was not carried down below the mud line a sufficient distance to prevent the teredo from destroying the piles below the concrete.

The question has been raised: Has any deterioration taken place in concrete structures standing in sea water in the harbors of the Southern States, where frost action is unknown? The most prominent concrete structure thus situated is the famous viaduct across the Florida Keys, built of Alsen cement, imported from Germany. It is possible that some of the members of this Society are in a position to give complete information regarding the action of salt water and the waves of the Gulf on this structure.

In order to guard against the disintegration (irrespective of its cause) of mass concrete placed *in situ* above low water, or to repair any damage that has been done, besides the cement gun process, various methods have been used, all based on the fundamental principle of using an impermeable material for the facing of the structure. Below low water, properly made block work has given most satisfactory results. Carefully made, fully cured pre-moulded concrete piles seem to resist the action of the sea and frost successfully. In Holland, hard, impermeable brick have been used to prevent any further damage to one of the breakwaters above low water. In England, the upper parts of massive breakwaters are mostly faced with granite or some other hard suitable stone. In Nova Scotia, both brick and pre-moulded blocks of concrete of small size were placed on the face of a concrete sea wall after the disintegrated concrete had been removed. A still more recent device is the use of hollow, vitrified, salt-glazed tile blocks filled with concrete after being put in place. Experiments thus far seem to have proved that:

"Vitrified salt-glazed tile is impervious to any deteriorating action of sea water, and has an effective structure against the battering of ice; it is so dense as to preclude the possibility of any water entering and freezing in it to the consequent destruction of the tile."

Though oiled concrete is being used as a water-proof material in certain cases, it is possible that the refuse, oil, gases, etc., discharged from certain classes of buildings, etc., might have a destructive effect

on the concrete foundation piles or other parts of the building, especially in sea water heavily charged with sewage. It is a well-known fact that concrete sewers will not perform their duty properly for any length of time unless they have a brick lining invert, over which flows the heavy sludge. In time of flood the surface water is so great as to dilute the sewage and prevent injurious effects. The writer would be pleased to hear opinions on this point, as it is possible that a destructive effect might have been caused by sewage in connection with one of the most serious cases in Boston.

Although poor results seem to have attended quite a number of the reinforced concrete structures standing in sea water in America, the opposite appears to have been true in foreign countries. Still, a few failures are on record as having occurred in England and Germany, due mostly to permeable concrete.

PART IV.—CONSTRUCTION AND MAINTENANCE COST OF REINFORCED CONCRETE DOCKS.

Cost of Construction.—To attempt any discussion of costs is always attended with danger to the one who does so, especially when such figures are "published cost". To lay the foundations for a discussion of this side of the question, the data in Table 2 have been collected from various publications, their real worth depending on the reliability of the published figures. The data cover some of the docks mentioned in Parts I and II.

The first cost of the various reinforced concrete dock structures at Port Talbot is stated to have been "no more than if they had been built of wood". Such a statement would not perhaps be true in the United States, on account of the large forests in this country.

Maintenance Cost.—A few figures covering the cost of maintenance of reinforced concrete docks in England are noted. In addition to what has already been said on the maintenance question, the cold storage wharf at Southampton is reported to have cost nothing to date for maintenance. On the other hand, the widened dock and the coal jetties are said to have shown considerable deterioration due to rusting of the steel, it having been improperly placed in these structures. It has been stated authoritatively that "while six to seven years is perhaps a rather short time in which to form any definite conclusions, the maintenance cost [of the above described Port Talbot docks] has been practically nothing".

The annual repair charges on the Purfleet coaling jetty (exclusive of the damage done at the time of the collision) for the first 9 years of its existence, are stated to have been but $50 per annum, which, based on its cost, $60 000, is less than one-tenth of 1 per cent.

TABLE 2.—COST OF CONCRETE DOCKS.

Location.	Type.	Cost per square foot.
Pier No. 8, Puget Sound Navy Yard...	Concrete columns. Steel deck-beams. Concrete deck-slab.	$3.11
Naval Station, Philippines............	Concrete columns. Steel deck-beams. Concrete deck-slab.	2.60
Balboa, Panama Canal................	Concrete columns. Concrete beams. Concrete deck-slab.	3.28
Oakland, Cal..........................	Concrete piles. Concrete beams. Concrete deck-slab.	3.27
Brunswick, Ga........................	Concrete piles. Wooden deck system.	1.40
Charleston Navy Yard, S. C...........	Concrete piles. Wooden decking.	2.60
United Fruit Company, Panama.......	Concrete-protected wooden piles. Concrete deck-beams. Concrete slab.	2.13
Brooklyn, average of two docks........	Wooden piles. Wooden caps. Concrete deck-slab.	0.90

Table 3, from data published by the Chief Engineer of the Mersey Dock and Harbor Board in 1910, gives some very interesting results as respects the cost of annual repairs to six of the reinforced concrete docks at Liverpool.

TABLE 3.—ANNUAL REPAIRS, LIVERPOOL DOCKS.

Dock.	Erected.	Cost.	Cost of repairs to date.	Average per year.	Annual percentage, based on original cost.	Remarks.
A	1899-1900	$24 000	$800	$80	0.0033	Deck for Cattle Wharf, Prince's Jetty (wood piles).
B	1904-06	63 500	375	75	0.0012	Floor or deck on Hennebique piles, Prince's Dock, West Quay.
C	1900	18 800	80	8	0.0005	Floor for wharf, Coburg Quay.
D	1901	3 700	25	3	0.0008	Floor for wharf, etc., Brunswick Half Tide Dock.
E	1908	17 500	......	......	....	Floor for wharf, etc., North Quay, Brocklebank Dock.
F	1908	163 500	......	......	....	Treble-story shed, South Quay, Sandon Dock.

Dock *A*. Subject to the effect of moist air arising from water below it.

Dock *B*. Subject to effect of moist air. Piles more or less submerged, according to water level.

Dock *C*. Complete reinforced shed and pile foundation: not sufficient time to form any conclusion.

If the results in Table 3 are true, it is no wonder that the English engineers report that their reinforced concrete docks cost nothing for annual repairs.

PART V.—GENERAL CONSIDERATIONS.

Opinions of Foreign Experts.—Although failures have accompanied the use of concrete in sea water and in the construction of reinforced concrete docks, it must be admitted that, if the Engineering Profession did not meet with a failure now and then, it would never acquire anything new, as it is through failures that it gains the most vital knowledge of engineering.

(*a*).—In discussing the question of concrete docks, a prominent New York engineer has stated that, of the large number of concrete docks which have come under his observation, the majority have been a success, though here and there he reports a failure due to poor construction and material, and not to defects in the design. It is authentically stated that the reinforced concrete docks at Southampton have shown no deterioration due to salt water action, except at the Southampton coal jetty. The engineers of the Liverpool Docks have been using concrete in connection with their work since 1872, apparently with great success. It has been stated on the best authority that in England the alternation of "dryness and wetness and fluctuations in temperature" does not appear to have affected reinforced concrete sea water structures adversely.

Mr. Henry Hunter, formerly Chief Engineer of the Manchester Ship Canal, England, states that "the concrete in the concrete lock built at Eastham is in better condition at the present date than the day it was deposited", and adds that, covering an experience of more than 30 years of placing concrete in salt water, he has known no failures in such work where the concrete has been properly mixed and deposited.

In discussing the concrete docks of the Port Talbot Dock Company, Mr. William Cleaver has stated that:

"While reinforced concrete requires extreme care, both in the choice of material and in the supervision of the workmanship, the results justify the extensive adoption of the material for dock work."

The exceptionally experienced dock engineer, Mr. Francis E. Wentworth-Sheilds, of the London and Southwestern Railway Company, has

stated that "if great care is exercised in making and placing concrete, an impermeable material will be obtained which can withstand the action of salt water". Mr. Sheilds has also said, "while many engineers were nervous about the life of reinforced concrete [for sea structures], he had observed that maintenance engineers were not so nervous as construction engineers". He is inclined to feel that there is nothing special to be feared respecting the life of reinforced concrete when used in marine work. Although, under certain circumstances, it is likely to deteriorate, he does not think it will do so from simply standing in sea water. He says that though in some cases deterioration has taken place at Southampton above low water, it has not done so below low water; that a 10-year-old reinforced concrete structure standing in sea water at Southampton is in perfect condition at the present date; and that, during the whole experience at Southampton, sea water does not seem to have produced any chemical or other deleterious action on the concrete.

Experiments by Mr. Baldwin-Wiseman, in 1907, in England, on the permeability of concrete, show that, if it is well made, it is one of the most water-tight materials known, and that it rapidly becomes less and less porous when water is forced through it.

Mr. V. de Blocq van Kuffeler, in summing up the experience in using concrete for salt water structures in Holland, says:

"A suitable mixture, very carefully manufactured, the use of a good brand of cement with trass, and setting in a moist atmosphere, are the most efficient means of ensuring the preservation of reinforced concrete in sea water."

Mr. I. Ho, one of Japan's expert harbor engineers, who used more than 1 200 mass-concrete blocks in one instance, none of which during a period of 10 years has shown the slightest signs of failure, states, "that whereas a good and proper cement is of consideration, the most important factor is the mode of fabrication".

In reviewing the successful experiences of some of England's leading authorities, the question of the chemical composition of the cement used does not appear to be given. Such information would be of great value, in order that engineers may know whether they use a cement especially manufactured for sea water concrete or simply the ordinary Portland cement, with or without puzzolana, trass, etc.

(*b*).—In discussing the deterioration of steel in reinforced concrete, by the action of sea water on ferro-concrete, provided the latter is

properly made, Mr. C. S. Meik, a prominent concrete engineer of England, says that such deterioration "is a negligible quantity". In support of this contention he cites the experience at Southampton, stating "that the exposed steelwork on a pile end that had been in the sea for 8 years was much corroded", whereas the bars in the body of the concrete, on being cut open, were found to be quite free from any rust and as fresh as the day they were put into the pile.

In connection with the building of some of the earlier concrete dock structures at Southampton, it appears that parts or the whole of piles not used were allowed to remain on the beach or shore, exposed to sea water, for some 7 years. At the end of this period the exposed steel had been badly rusted and deteriorated, whereas the part which was embedded in concrete was found to be in fine condition, practically as good as the day it was placed in the concrete. Still, concrete piles lying on the beach are not in the same position as concrete piles subjected to shocks in a dock.

Though the first jetties, built 11 years ago in Southampton water, are in excellent condition at present, the steel in another jetty at the same location has deteriorated, due to electrolytic action.

It is generally accepted by all English authorities that no deterioration takes place in steel when well embedded in the concrete.

(*c*).—In speaking of reinforced concrete when used in marine work, Mr. Wentworth-Sheilds says "it will stand a wonderful amount of shock, and bending due to shocks, if a wooden fender is interposed". At a more recent date, Mr. Sheilds remarked: "On the other hand, reinforced concrete would not bear being knocked about by heavy ships, and where a structure was subjected to severe blows of that sort it was not easy to find anything better than timber, * * * but, when used at the right time and in the right place, reinforced concrete was a valuable material to dock engineers." From the leading position Mr. Sheilds occupies among the dock engineers of England, it would be of interest to know just what distinction there is between "a wonderful amount of shock" and "subjected to severe blows".

As an axiom: whatever system or design is adopted for a reinforced concrete dock, in no manner whatsoever should a vessel be allowed to rub against the main piling of the dock. The dock should always be protected by a system of fender-piles.

(*d*).—As respects the resistance of a concrete pier or dock, when under such treatment as the Purfleet pier was at the time it was rammed, in 1904, Mr. Meik, engineer in charge of its construction, states that "the vibration was so great at the time of the collision that they thought the entire pier would collapse, but that its elasticity was most satisfactory, due no doubt to its horizontal concrete decking". Mr. Meik also says that the vibration of a concrete pier supported on piles is nearly as great as in a pier made of timber piling; but this has no particular effect on the structure, judging by the experience gained with the Purfleet pier at the time it was rammed.

In speaking of the Port Talbot docks, Mr. A. E. Carey has stated: "If the structure [reinforced concrete dock] was properly designed and built, its stability and life were assured, the only serious drawback being the difficulty of repairing damage due to collision."

But how often do collisions happen?

Mr. Bryson Cunningham, of the Chief Engineer's staff, of the Port of London Authorities (Mr. C. R. S. Kirkpatrick, Chief Engineer), recently stated that the art of building reinforced concrete docks in England has attained a degree of perfection greatly in advance of early experimental work. If their early experimental docks are still doing good service, will not their more recent docks become structures of an engineering and commercial success, thus justifying the American engineer in recommending concrete docks as long as they are built in a manner to guarantee impermeability and non-deterioration of the concrete?

Concrete Breakwaters.—As the application of reinforced concrete to dock construction has been developed almost entirely since 1900, some of the most conservative engineers may not feel that sufficient time has elapsed to judge correctly as to the merits of using cement and placing concrete structures in sea water, and as to the advisability of adopting reinforced concrete as a coming type of dock structure. As respects the first point, the prolonged and successful use of mass concrete by foreign countries in breakwater, tidal, and graving dock work would in itself appear to be sufficient answer to all such skepticism.

Though an extensive treatise might be written on concrete breakwaters and their construction, including shore protection, those phases of the use of concrete in sea water are supplementary to the principal subject of this paper. There is apparently hardly a leading seaport or

a maritime nation outside of the United States that has not made a wide, extensive, and successful use of mass and reinforced concrete in the development of harbor and shipping facilities.

Surprising as it may seem, a number of large concrete breakwaters have been in existence in Japan for more than 18 years. As a matter of fact, the use of concrete in the harbor work of Japan is far in advance of American practice, being apparently on the same high level as in England and other European countries.

Mass concrete has been used very extensively in Belgium and Holland for sea water structures for years, and has given the best of results. In fact, some of the concrete sea walls in the latter country, built in 1867-77 and earlier, are so old as to be called ancient, and have as yet shown no signs of being affected by the action of sea water. Perhaps such a statement needs to be qualified, because some of the principal harbors of Holland are some distance from the sea, and are in fresh or brackish water.

Concrete was used by the Romans and Carthagenians in ancient times. Though it fell into disuse for many centuries, it came into use again in 1840-50. To this day, sea walls built of puzzolana and lime cement by the Romans are in existence in Italy.

The Italian engineers report that Portland cement concrete with an addition of one-eighth to one-tenth by volume of puzzolana gave no signs of disintegrating in salt water, even after an exposure of 30 years in the harbors of Genoa, Civita Vecchia, Naples, etc.

At several places on the Italian coast concrete-faced breakwaters have been in existence since 1880, in most exposed positions, costing but little for repairs and maintenance, though subject to the high seas and heavy blows of the Mediterranean.

English engineers were using mass concrete in their tide locks and in the construction of massive breakwaters along the coast of England as far back as 1871, if not earlier. The fact that they have continued to use it more extensively each year, even to building vast reinforced concrete structures standing in sea water during the past 15 years, would appear, in spite of some failures, to be sufficient answer to any doubts the American engineer may entertain on the subject.

At Colombo, India, a concrete breakwater, finished in 1885, showed no failures above or below water at the end of 22 years. As stated above, some of the massive concrete breakwaters of the world have a

hard stone facing, or are built of concrete blocks, with or without a stone facing.

The reasons that enable the foreign engineers to accomplish such lasting results with concrete is no doubt due to the fact that they, together with the foreign chemists and cement manufacturers, long ago learned the secret and acquired the art of manufacturing and using concrete in sea water structures. Though the American engineer excels the foreign engineer in certain lines of his Profession, it must be admitted that, so far as using cement, and hence concrete, in sea water structures, the engineers of the leading European countries and of certain parts of South America are many years in advance. The American cement manufacturer, the chemist, and the harbor development engineer cannot long remain in such a position without reflecting on their ability as experts in their respective lines of work in the minds of their foreign contemporaries.

Part VI.—Conclusion.

In view of the marked success obtained by foreign engineers in the use of concrete for sea water structures, when the execution of such undertakings has been placed in the hands of intelligent, skilful, and experienced men, the American engineer who denies the possibility of making a successful use of concrete for structures standing in sea water puts himself in a questionable position. He thereby confesses either his lack of a world-wide knowledge on the subject, or his inability to carry out properly such classes of construction work to the same successful conclusions as his foreign contemporaries have been doing for years past. Such a confession would seem to indicate a lack of foresight and ultra-conservatism as respects the use of cement subject to sea water conditions, on the part of the American cement manufacturer, chemist, and concrete engineer.

The American engineer who assumes a skeptical attitude toward the practicability and commercial success of reinforced concrete docks has standing before him as silent testimony of their worth and practicability such a vast number of foreign reinforced concrete docks—several about 20 years old and yet in excellent condition, with still more massive structures being built each year, some of them costing less than wooden docks, if reports are true, and most of them saving their owners large sums annually on account of their low cost of main-

tenance and repairs, with no rebuilding, as with our 15-year creosoted wooden pile docks—that the grounds on which he stands become somewhat untenable. It is true that all American cements are not as yet wholly suitable for sea water purposes, and perhaps equally true that each and every American concrete engineer has not hitherto insisted on the proper placing of concrete in salt water structures, not fully realizing the importance of the fundamental principles of the use of concrete in sea water, due to the hitherto limited call for such types of structures in America, lumber having been so plentiful and cheap.

Though reinforced concrete docks have been in existence for more than 15 years, and operated successfully, the same old theorems are still put forth in opposition to them and to the practical experience gained during this period. Possibly these same theories will continue to be advanced against the use of concrete in dock work and other sea structures by the most conservative of our leading engineers, though others, guided and profiting by the experience already gained in such uses of concrete, will continue to expend large sums of money in the further development of such structures.

From a prolonged study of reinforced concrete dock construction, as carried out in foreign countries, it would appear that, in spite of early doubts and skepticism, the success in the use of reinforced concrete in dock work obtained by foreign engineers has swept away all such doubts and skepticism. If these are not facts, why are foreign countries and certain ports in America, including the United States Government, expending vast sums of money in building reinforced concrete docks and in other uses of concrete in harbor development and sea protection work; all "in spite of prejudices which leading engineers [psychologically] have against any new type of construction".

Although the average American contractor may look upon concrete as just so much cement, sand, and stone, or gravel, to be thrown together and dumped into the forms in the quickest possible time, without any regard for the fundamental principles underlying reinforced concrete construction, a commercial proposition purely, such an application of reinforced concrete to dock work will most certainly spell disaster long before the structure is completed.

In spite of its apparent simplicity on dry land, the use of reinforced concrete in dock work calls for more than mere brawn and

muscle. It is a class of construction work especially adapted to the broad knowledge, experience, and deep study of the trained engineer in association with an organization well skilled in the handling of concrete in sea water structures—"a field of engineering in which reinforced concrete will prove to be the most permanent and economical, as it has in the building of bridges, etc."

DISCUSSION

Mr. Muirhead.

J. H. H. Muirhead,* M. Am. Soc. C. E.—In regard to concrete dock construction, the speaker always looks on Para as one of the examples of what not to do. While there, he calculated that, approximately, £5 000 000, sterling, had been spent for the accommodation of five ships. Of course, that calculation is not altogether fair, yet it is not unfair, because it illustrates a system of finance radically different from that which is customary in the United States. In fact, it is a system that would not be tolerated in American undertakings. It is true that comparatively little more money was required to complete the work, which now accommodates a large number of ships, nevertheless, the initial outlay was enormous, and quite unwarranted by the results.

The speaker regrets that as a Society we spend too much time in the discussion of details and do not study the general schemes as we ought. He knows of no justification for the construction of such expensive port works at Para, beyond the fact that the concessioners and contractors made a profit. True, the civil engineers made something, also, but not by any means enough to compensate for some forty lives lost by yellow fever. Quite a brisk shipping business has been done in materials for the improvement of the City of Belem de Para itself, but that is temporary.

The only trade that Para has, or is ever likely to have, is rubber. Are not our friends, the mechanical engineers, doing all they can to devise substitutes for rubber? It is being planted in many countries. The conditions under which it is obtained in Para and the neighboring States are so bad that it is a very precarious industry. The fluctuations in price are great, taxation is unreasonable, and "ladrones" are only too common.

The chief work at Para consisted in a bulkhead wall, built on the "slice block" principle, extending along one bank of the River Amazon, in front of the city, which provided 35 ft. of water alongside the wall. The speaker cannot see that this system was at all suitable for the conditions to be met on the Amazon. The glaring defect was the absence of stone with which to make concrete. Cement was imported from England and sand of excellent quality was obtained a day's journey down the river. The blocks were made of cement and sand only.

At the same time, clay suitable for brick-making was being dredged immediately in front of the wall and disposed of at sea or in the deep parts of the Amazon quite a distance away. From experience gained by the speaker in Kingston, Jamaica, and from the opinions of men who worked there when that city was being rebuilt after the earthquake, much the finest concrete-making material one can have is broken brick. On one occasion, the speaker had to remove some broken-brick concrete,

* Glasgow, Scotland.

Mr. Muirhead.

and although it had set only a comparatively short time, the difficulty of removal has given him a very great respect for concrete of this class. He has never heard of making brick and deliberately breaking it up when almost new as a substitute for gravel or broken stone. Under given conditions, the experiment should have been made. The design contemplated 30-ton blocks. Had they been made in the proportion of 1 part cement, 2 parts sand, and 4 parts broken brick, a great saving would have been effected over 1 part cement to 2 or even 3 parts sand.

In some places, stone for ballasting was necessary. In these cases, a very fair grade of iron ore was used. This was also a waste of good material. Again, the speaker would suggest broken brick, even when the brick has to be made and then broken by mechanical means.

Mr. Barney.

W. J. Barney,* Assoc. M. Am. Soc. C. E.—This paper serves at least one purpose, in that it places data relating to dock construction before the members of the Society. The great problem in such construction is concrete *versus* timber, and Mr. Taft has gathered together original references and made suggestions as to where information thereon may be obtained. The paper represents weeks of compilation in the library and by correspondence, and involves a great deal of hard research work. The speaker hopes that it will be given the benefit of careful discussion of the many points involved in concrete *versus* timber pier construction and a combination of concrete and timber piers.

In Portland, Ore., it was a great question whether concrete or concrete and timber combined should be adopted. The city is in a timber district, yet the necessity of avoiding fire risks seemed to compel the use of concrete, and a combination of timber below the water line with concrete above, was used in order to avoid such risk and, at the same time, keep down the cost. To the speaker, the great question in such matters has always seemed to be the "commercial life" of the piers; that is a vital factor which is frequently ignored. In foreign ports, pier construction has had practically to be dynamited out, because its "commercial life" had passed away and new construction was necessary for larger vessels and changed conditions.

These questions should be considered and discussed more widely than they have been, particularly in connection with this paper.

Mr. Stern.

Eugene W. Stern,* M. Am. Soc. C. E.—There are some general considerations in this paper which it might be well to review carefully.

It will doubtless be conceded that it is desirable in many cases to substitute a more durable material than wood for docks and piers, provided the cost is not prohibitive, and its permanency in sea water can be assured.

* New York City.

Mr. Stern. Mr. Taft has gone to a great deal of trouble to gather information and present it in concise form, but he treats rather indifferently the fact that in the United States there have been some serious and disastrous experiences with concrete structures in sea water. He cites many cases in which concrete has stood well, but does not dwell sufficiently on the failures.

Some 10 years ago, the speaker's attention was first called to the disintegration of concrete in sea water on some pier work in the lower part of the Hudson River. Since that time he has been following this subject carefully, with the result that he is convinced that there is as yet considerable uncertainty as to the action of concrete in sea water, and until that uncertainty has been removed, engineers must go slowly.

At Atlantic City quite a number of concrete structures are exposed to the action of sea water. A few years ago, the speaker investigated the condition of almost all these structures. It was just about the time when the large cylindrical concrete piles were being sunk for the Boardwalk, but the speaker has not been able since to examine these particular piles. Every structure in Atlantic City that the speaker examined, from one end to the other, which came in contact with sea water, showed more or less deterioration, and it is believed that any one with open mind, who went there to investigate, would be similarly convinced. The reinforced concrete structures built over the sea showed, not only the effects of frost and the action of waves, but also chemical disintegration.

There are at least three kinds of action which may cause deterioration in concrete structures exposed to sea water:

1.—The effect of the impact of the waves;
2.—The effect of frost;
3.—The effect of chemical action.

At Atlantic City, it seemed to the speaker that a great deal of disintegration was due to chemical action. In places the concrete had cracked so much that the salt water had penetrated to the reinforcement, which had rusted badly. The concrete had also swelled and cracked off in places. In one of the large piers, built a few years ago and projecting into the ocean, the piles had shells of boiler iron, 18 in. in diameter, riveted together. These had been sunk and then filled with concrete. At the time the speaker examined this pier (probably then about 10 years old) nearly every one of the cylinders had been split open by some great internal pressure, due without doubt to chemical action having taken place in the concrete, caused by sea water entering through the joints of the shell, which were not caulked. Many vertical joints in the iron shells were burst open

Mr. Stern.

and the rivet heads sheared off. The joints had opened to such an extent that part of the concrete filling could be raked out, and was found to be almost in a powdered condition.

A series of valuable experiments is now being made by the Aberthaw Construction Company, of Boston. About 5 years ago, twenty-four square piles, about 16 by 16 in. and 16 ft. long, were prepared. These were placed in the sea at the Charlestown Navy Yard, their tops being about 18 in. out of water at high tide, and their bottoms about 4½ ft. under water at low tide, the mean tidal range being 10 ft. These piles were examined in December, 1913, after having been in place about 5 years. The published report of the Aberthaw Construction Company shows that only one of these twenty-four piles did not show disintegration of some kind. One pile was eaten through; others were eaten into 4 or 5 in.; others showed slight defects on the edges only.

The one pile which stood the test was made of German iron-ore cement of a rich wet mixture, namely, 1:1:2. Other piles made of the same mixture, but of a native cement, stood well, but showed some disintegration. Now, the question arises, what will the result be after 20, or 30, or 50 years?

These tests prove that the cement chemists and manufacturers must give the engineer a cement that will better withstand sea water.

It is all very well to say that concrete docks should be built. The speaker believes in having permanent dock structures, if they can be built economically, and with reasonable assurance that they will last, but there must be this assurance before such construction is undertaken on an extensive scale.

There is no doubt that something better than wood is often very necessary. On the Pacific Coast, the speaker has seen large piles which have been completely cut through by marine borers in a few months.

Engineers are very anxious to get something better than wood for this purpose, and if German cement makers can produce a cement which, at the end of 5 years, shows no disintegration in sea water, those of America should be able to do as well.

The author brings up the question as to the effects of frost on masonry structures in the Great Lakes. The speaker has observed that in the lock at Sault Ste. Marie, on the Canadian side, the walls built of limestone show, at the water line, disintegration due to frost. The stone seems to be rather porous, and in some places the cavities are several inches in depth. There is no doubt that, wherever concrete is porous, frost action will be more or less harmful, so that it is necessary to get a dense mixture for concrete exposed to frost action, whether in fresh or sea water.

Mr. Coombs.

R. D. COOMBS,* M. AM. SOC. C. E.—If the speaker remembers the various reports of the Masonry Committee of the American Railway Engineering Association, the gist of all of them has been that, for waterproofing, or preventing decomposition in salt water, the main thing is to make a very dense concrete, and that is practically the summation of all other reports.

Mr. Pemoff.

JOEL J. PEMOFF,* M. AM. SOC. C. E.—The Dock Department of New York City has been building sea walls of concrete blocks set in place under water for the the last 40 years. The cement used in these blocks is such as is usually specified for first-class work. The Department has never met with failure, or even deterioration, in any portion of this work, so far as could be ascertained by submarine examinations made by divers. Only in a very few places can disintegration be noticed, and those places are between high and low water.

The great success of the Dock Department with its submarine work is due to the mixture used, 1:2:5; to the dense surface produced on the blocks; to the fact that Portland cement of the very best quality is always specified and obtained for this work; and probably also to the condition of the waters in this harbor, which carry a great deal of fine matter in suspension, tending to form a slimy coating on the blocks and thereby increasing the imperviousness of the outer skin.

One of the causes of disintegration between high and low water is the fact that this portion of the work is usually deposited *in situ* while the tide is low, and the early removal of the mould boards allows the sea water to reach the concrete before it is sufficiently seasoned to withstand its effects. The disintegration of exposed concrete surfaces in New York Harbor is rather the exception than the rule.

Mr. Goldsmith.

WILLIAM GOLDSMITH,* ASSOC. M. AM. SOC. C. E.—Reinforced concrete piles, as well as a new type of reinforced concrete bridge resting on piles, have been used in many places throughout the Philippine Islands. These piles are usually 16 by 16 in., approximately 40 ft. long, and have a 1½-in. steel pipe running through their centers so that they may be jetted into place. Pantal and Calmay Bridges, being, respectively, 300 and 600 ft. long, are on piles of this type.

There are five piles in a bent, and the bents are 20 ft. from center to center. Reinforced concrete girders rest on these piles, the bridge being designed to support a 12-ton road roller.

These bridges have been in use about 5 years, and many more have been built. They have stood the test of salt water very well, and show no deterioration due to this cause. The cement used in their manufacture is "Green Island cement," made in China, the concrete mixture being 1:2:4.

Wooden piles in the same locations have lasted only 2 or 3 months. The teredo is very active in these waters, a wooden pile bridge being

* New York City.

hardly completed when the work of the teredo would be so evident as to make the structure dangerous. It would seem, therefore, that wooden bridges have no place in Philippine waters and that concrete is the only lasting material which can be used for such structures. Mr. Goldsmith.

The type of bridge referred to is also particularly adapted for Philippine construction on account of its low cost in comparison with that of arch bridges of wood or even of steel, the life of which is also of short duration.

Great care is taken in the testing of Portland cement in the Philippine Islands. The "standard method for analysis of cement, proposed by the Committee on Uniformity in Analysis of Materials for the Portland Cement Industry," is used. In the field, each district engineer has sufficient apparatus to make 24-hour and 7-day tensile tests, thus giving an independent check on the cement inspection.

DeWitt C. Webb,* M. Am. Soc. C. E. (by letter).—This paper is a valuable compilation of information in regard to concrete structures in water, particularly in sea water. Mr. Webb.

Some of the author's statements in regard to the deterioration of concrete in Boston Harbor are not in accordance with the writer's observations. The author, in laying stress on the action of frost in disintegrating concrete, states: "In Boston Harbor 12° below zero is not uncommon." An examination of the records of the Weather Bureau for the 41 years from 1873 to 1913, inclusive, shows only two days during that period when the minimum temperature was 12° below zero or lower. During the same period, there were only 121 days, or an average of less than 3 per year, when the minimum temperature was zero or lower.

The impression conveyed by the author, that frost has been the principal cause of the disintegration of concrete structures in Boston Harbor, appears to coincide with the general opinion. During the past 5 years, the writer has had opportunities for careful examination of many of these structures, and has supervised repairs to several of them. The result of this experience has been the formation of a very firm opinion that although frost and wave action have had some effect, the principal destructive agent has been chemical action. If frost were the most active destructive element, the concrete behind and adjacent to the disintegrated areas would, it is thought, be sound and hard. In several instances this was found not to be the case. On the contrary, the concrete which was cut away to permit a new facing to be applied, was soft, and, so far as could be determined, this condition extended through the entire mass. Pockets and streaks of a white putty-like substance were found in the concrete removed. This material was chemically active to such an extent that it burned the hands of the workmen. Neither the appearance of this substance

* Boston, Mass.

Mr. Webb.

nor the manner in which it occurred suggested that it might be laitance, nor does its chemical composition, as given below, appear to indicate such an explanation:

	Percentage.
Lime (CaO)	31.11
Carbon dioxide (CO_2)	9.63
Silica (SiO_2)	8.87
Sulphuric anhydride (SO_3)	6.68
Magnesia (MgO)	5.35
Iron and alumina oxides	4.38
Insoluble residue	5.86
Combined water	12.05
Moisture	13.66
Salt and undetermined	2.41
	100.00

The cement used originally in this work was a well-known brand, and the records of the chemical analyses show the following average results:

	Percentage.
SiO_2	22.50
$(Al_2Fe_2)O_3$	9.55
CaO	61.98
MgO	2.80
SO_3	1.33

It appears to the writer that these statements tend to confirm the theory that the chemical action of sea water on concrete is due to the flux and reflux of the salt water in and out of the voids, gradually causing magnesia hydrate to form and abstracting the lime from the cement. It is only when concrete is porous and exposed to the unbalanced or intermittent action of sea water that trouble occurs, as it is the continuous percolation and renewal of the sea water which does the mischief. For this reason concrete below low tide is usually secure.

The remedy is not chemical, but mechanical. The sea water must be prevented from penetrating the concrete. This can best be done by the use of a dense, rich mixture, placed wet. It is desirable, when possible, to permit the concrete to harden before exposing it to sea water, but this is not absolutely essential, as can be proved by concrete structures in Boston Harbor which were not pre-moulded or built inside of coffer-dams.

The damage to the concrete sea wall at Lynn, Mass., mentioned by the author, is believed to be due almost entirely to the pounding of the waves and wave-borne materials such as driftwood, pebbles, and larger stones from the beach. The repairs by the cement gun, when

last observed, did not appear to the writer to be entirely effective. This might be expected, as the repair work was largely near low-water mark, and the rising tide would bring the repaired areas within reach of wave action shortly after the cement-gun plaster was applied. Mr. Webb.

The writer believes that concrete, properly proportioned, mixed, and placed, is an entirely suitable material for structures in sea water in any climate. It is desirable, however, to provide a thin facing of stone, hard brick, or similar material between high and low water, where it is exposed to destructive action from waves and wave-borne objects.

JOSHUA F. RAMSBOTHAM,* M. AM. SOC. C. E. (by letter).—Mr. Taft is to be congratulated on the clear and concise conclusions which he has drawn from his evidently deep study of the experiences of all nations in the use of ferro-concrete as applied to marine works. Mr. Ramsbotham.

The writer agrees with the conclusions drawn, and would like to observe that, as far as he is aware, there is no other paper or book which covers, to the same extent, the large range under review.

In regard to the use of cement under sea water: It was the writer's privilege to remove 68 404 cu. yd. of concrete, made in the earliest days of Portland cement concrete, in the Port of Liverpool, while acting as Resident Engineer on the Brunswick Dock Extension. The concrete, which was almost 40 years old, was perfect, and in no single instance was there any sign of decay or shoddy work. During that period of 40 years, the work had been under sea water, the walls, etc., in the first instance having been built in the dry.

It is the writer's firm conviction that in building graving docks, walls, etc., it is unwise to place freshly mixed concrete in water, for the reasons given by Mr. Taft.

It is doubtful whether true reasoning has been observed as regards the Cattle Wharf at Liverpool, the deck being a considerable height above the water, and the writer cannot help thinking that "bloom" on the steel may have been the cause necessitating repairs. The author has not mentioned this important factor in the problem, but it is an absolute necessity that all bloom, dirt, or rust, should be removed before concrete is placed in the forms; if a skin of bloom is on the steel, the cement does not reach the metal, and trouble may be expected in the future.

The writer does not agree with the four conclusions drawn by an English engineer, as ferro-concrete work is certainly not indestructible, either from fire or shock, and, further, it is by no means easy to repair, in fact one can hardly conceive a more difficult job than the repair of a partly destroyed ferro-concrete structure; it would resolve itself into points, wedges, and hack-saws, and under water it would surely be most difficult.

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* Melbourne, Victoria, Australia.

Mr. Ramsbotham. Ferro-concrete has its proper sphere of usefulness as applied to marine works, but, at the same time, and surely to a large extent, the question is resolved by prime cost and permanency. At Liverpool, ferro-concrete was approached by Mr. Anthony G. Lyster, then Engineer-in-Chief to the Mersey Docks and Harbour Board, with care and caution, and all the work was done in such a manner that it was under review, and under no extreme circumstances could it interfere with the traffic of the Port. The policy was to make M. Hennebique not only responsible for the design, but also solely responsible for carrying out the work in a suitable manner. At the same time, Mr. Lyster had his own independent staff, who supervised and checked everything.

The writer maintains that the design cannot be separated from the execution, and that a dual control results in extravagance and inefficiency, possibly accompanied by disaster.

Mr. Davis. CHANDLER DAVIS,* M. AM. SOC. C. E. (by letter).—This paper, covering, as it does, the most important structures of the world, is most interesting and valuable, and deserves the careful study of engineers who are interested in this subject.

The most vulnerable part of maritime structures is that which lies below high water, as the materials are exposed to the action of water, which, as a rule, is not over clean. Most harbors are near or in large cities which generally empty their sewers into the rivers or bays on which they are located, and thus docks and other structures are exposed to the action of, not only sewage, but also refuse discharged by factories, chemical works, and similar plants. These local conditions must be taken into consideration in drawing the specifications under which the materials are furnished and inspected.

The laboratory testing of the materials is as important as the field inspection, and should not be neglected. It is not sufficient to see that the plans are properly carried out, but every effort should be made to see that the supplies conform to carefully drawn specifications. These should read so as to cover every test to insure the use of only first-class materials, suitable for the work in view, and acceptable to the engineer. The contractor should be obliged to have sufficient material on hand to permit of a thorough test and inspection before it is used in the work. This applies especially to cement, and none should be permitted to be used under the 24-hour test. In fact, if it is possible, the cement should be held 28 days, or even longer, in order to determine its increase in strength over the 7-day test. This is important if the concrete is to be used in water, and especially if an impervious wall is required, such as would be the case in the construction of a dry dock.

* New York City.

Mr. Davis.

Many specifications state the component parts of which the materials should be made. This, in certain instances, can be done, but, on the whole, it is safer to limit the quantities of those ingredients which, necessarily, must be a part of the finished substance, but are to an extent injurious, and leave the rest to the manufacturer. By using only well-known and thoroughly tested brands, in which the injurious components have been reduced to a practical minimum, the engineer obtains the quality and durability for which he is looking. The fact that known and tried supplies are being furnished does not relieve the engineer, however, of his duty as an inspector, and he must still insist on clean, pure materials, free from all foreign matter and substances which are likely to injure the material and impair its strength and durability, and that they should be free from adulterants, which, perhaps are harmless in themselves, but have no special value.

The engineer should insist on the use of the best and only first-class materials, and should see that all analyses and tests are carefully made by competent assistants, reserving for himself alone the right to accept materials, which, although they do not quite conform to the specifications, may, in his opinion, be satisfactory.

Reinforced concrete has been used extensively throughout the world, and with great success. There have been failures, but the general results have been good and satisfactory.

The use of concrete and reinforced concrete in dock work is increasing annually. It has been applied to quay walls, storage and cold storage warehouses, piers, and piles. Great care, however, must be used in making, manipulating, and placing the concrete, for, if used in salt water, it must be made impervious; this also applies to concrete placed in fresh water. In order to obtain impervious concrete, the percentage of mortar in the mixture should be larger than that of the voids in the stone, and the quantity of cement in the mortar should never be less than one-half of the quantity of sand. If this is done, the mortar will fill all the voids in the stone and properly cover it with mortar, the concrete will then set properly, the mass will be thoroughly cemented together, and the structure will be impervious to water. In certain instances the face of the work may be protected by a layer of richer concrete. Assuming a quay wall to be built of 1:2:5 concrete, the proportions in the outer 18 to 24 in. which is exposed to the action of the water, would be 1:2:3. This richer concrete will resist the flow of water through it, and the weight of the whole wall will carry the superimposed loads and resist the pressure due to the filling behind it. The engineers of the Liverpool docks have used concrete in salt water for more than 40 years, and their experience has been that it will resist the action of salt water if properly made and deposited, *en masse, in situ.*

Mr. Davis.

The Dock Department of the City of New York has also been using concrete in the sometimes salt, sometimes brackish, but always dirty, waters of the North and East Rivers for more than 40 years. An examination of the sea wall shows no deterioration, the concrete being as firm and solid as the day the blocks were set. This wall, however, was built of blocks manufactured on dry land, and not placed in the work until they had hardened sufficiently to be handled with safety. Each block weighs about 75 tons. As the concrete is protected, from a level 2 ft. below mean low water up, with a granite veneering, there is no way of determining how the material would have worn if it had been subjected to contact with the ice.

Pier No. 1 (New) North River, which was built about 40 years ago, rests on concrete piers and arches; only the outer faces are protected by granite. The arches under the dock, however, are badly pitted and scarred at about low water; this concrete was laid *en masse, in situ.*

In the early Seventies this Department had little success in depositing concrete under water directly in the work, and that method was soon abandoned; then the old scheme was taken up, of building the blocks on dry land and setting them in the work with a derrick.

A very interesting and instructive example of the failure of a concrete graving dock occurred in Aberdeen, Scotland. The dock was opened in 1885 and immediately showed signs of leaks, which gradually grew to be so bad that, in 1887, a thorough examination of the structure was made. Mr. Philip J. Messent, who was called in as consulting engineer, took samples from various parts of the dock, and in his report states that the deterioration existed chiefly in those sections where the proportions of sand to cement exceeded the ratio of two to one. In some places the mortar was a mixture of more than 1 part of cement to 3 parts of sand. Mr. Messent also found that the mortar plastered on the surface of the concrete, which was mixed in the proportions of 1 part of cement to 1½ parts of sand, had hardened thoroughly, and was in very good condition, as was also the concrete mixed 1 part cement, 2 parts sand, and 4 parts broken stone. The failure of this dock was due undoubtedly to the laxity of the inspectors on the work, who permitted the use of a mixture which, although composed of cement, sand, and broken stone, could not be called concrete. As the concrete was not impervious, the salt water penetrated it quite freely, and changed the composition of the cement, which resulted in failure. Samples were cut out of the concrete and sent to Professor Brazier, of Aberdeen University, to be analyzed. It was found that the deterioration was due to the chemical action of the salt water on the cement. Professor Brazier says:

"The analyses of the series of decomposed cement show a remarkable difference to the original cement, inasmuch as that in all these samples there is found a large quantity of magnesia, and a large

proportion of the lime in the form of carbonate. I believe this alteration is brought about entirely by the action of sea-water upon the cement. There is no other source for either the magnesia or the carbonic acid."* Mr. Davis.

Every engineer should direct his energy toward obtaining an impervious concrete, especially in the case of graving docks; and, as it is possible to make concrete water-proof, and such concrete has proven to be the more lasting, the extra time devoted to obtaining this end is well spent.

The dry dock built by Luigi Luiggi, M. Am. Soc. C. E., for the Argentine Government, at the Naval Station at Bahia Blanca, was put in commission in 1902. This structure is perfectly tight, and has not shown any signs of deterioration or leakage.

The greatest care, however, was taken in making and placing the concrete in this work. Each delivery of cement was thoroughly tested before it was used. The proportions laid down by the specifications were strictly adhered to, and the joints between the various layers of concrete were carefully manipulated at the beginning of each day's work, so as to avoid all chances of leakage at these points.

In order to insure a good concrete, the proportions having been determined, the cement, sand, and broken stone must be carefully gauged and thoroughly mixed, so that each particle of stone will be surrounded by a coating of mortar. The mixing is best done by a machine, if large quantities of concrete are to be laid. There are many kinds of such machine on the market. All have their good points, and the engineer should decide for himself which mixer is best adapted to his particular work.

The consistency of the concrete, again, is a matter of opinion and judgment, some engineers prefer it very liquid, others require it to be dense. In the latter case it is essential to tamp the concrete thoroughly after it has been laid in place, and the layers should not be more than 6 in. thick. In the former, the concrete will take care of itself, and the tamping is not so important. In either case, whether wet or dry concrete is used, great care must be taken to make a strong and tight joint between old and new work. To do this, the surface of the old concrete should be roughened by picking, and thoroughly cleaned and moistened; a layer of rich mortar, from 2 to 3 in. thick, should then be placed on the clean surface and the new concrete should be thoroughly tamped into the fresh mortar bed. In this manner, a good junction between the old and new work will be obtained, which will not cause trouble.

It is frequently necessary to lay concrete in cold weather. Although it is not advisable to do this, it may at times be necessary to push

* The results of the investigations will be found in full in *Minutes of Proceedings*, Inst. C. E., Vol. CVII.

Mr. Davis. work to completion in spite of the low temperature. In order to prevent the concrete from freezing, some precaution must be taken, either by maintaining fires in stoves or by raising the temperature with steam coils, according to the location and magnitude of the work. On the other hand, it is just as important to protect the concrete from the direct rays of the sun, and prevent the water in the mixture from being evaporated during very hot weather, or the concrete will not set properly, will not gain its full strength, and the surface will be likely to crumble. Frequent watering of the surface of the concrete with a spraying nozzle, and covering the surface with moist sand, saw-dust, or canvas tarpaulins, will prevent the water from being evaporated, and the concrete will set properly.

Frozen concrete will in time thaw out and eventually set. The following may be of interest, as it shows what may happen to frozen concrete; although it is not intended to indicate by this experience that concrete should be allowed to freeze, even when being built in as large masses as the blocks mentioned. It was found necessary to move some concrete blocks, which were about 10 ft. 6 in. by 6 ft. 8 in. by 8 ft. and had been built in freezing weather. According to custom, the work of concreting was stopped as soon as the temperature in the moulds fell to 36° Fahr. The blocks had been completed and allowed to stand for 6 or 8 days, when it was decided to shift them to make room for more work. Chains were rove through the chain holes, and, as the attempt was made to lift the blocks, the chains cut through the concrete. It was found that small crystals of ice had formed in the concrete, and that the process of setting and hardening had stopped. The remaining blocks were left undisturbed until the following spring, when they were handled and set in the sea wall without further trouble. These blocks behaved well and showed no sign of having deteriorated, although built at the same time as the two which broke in handling during the cold spell. This experience does not indicate that concreting should be carried on during cold weather without some precaution being taken to prevent freezing, but that, if concrete, in large masses, once frozen, is left undisturbed, it eventually may thaw out and perhaps set properly. No samples were taken to test for hardness or porosity; in this case, weight was the main requirement.

It is impossible to expect good and permanent results if poor materials are used in concrete, even if laid with the greatest care, as was the case in the following illustration: A reservoir, part of a private water supply, was discovered to be leaking; in fact, it failed suddenly, and would not hold water. The conditions revealed by an examination made it all the more surprising that the structure was ever water-tight. It was built partly on rock and partly on a very hard clay. Its floor consisted of about 5 in. of concrete made with

sand, gravel, and first-class American Portland cement. The sand and gravel contained a very large percentage of clay, loam, and foreign matter, nearly 50% of their ingredients being injurious to the concrete. The old floor was easily removed, being readily picked up; in fact, the clay under it was considerably harder than the concrete; its appearance at first glance was good, it was apparently smooth and hard; on a closer examination, however, a large number of soft spots were located, and the entire surface was found to be pitted. Mr. Davis.

The sand used was reported to have come from a sand bank which furnished the materials to be used in the repair work. This can readily be believed, as the sand and gravel, after being screened and washed, yielded about ¼ cu. yd. to the load of 1¼ cu. yd., and in one case 1 cu. yd. of sand shrank to about ⅛ cu. yd. after being screened and washed.

It is the use of such materials that condemns concrete, and probably is the cause of most of the failures which are reported from time to time. Absolutely clean materials should be insisted on, and none other permitted to enter into the mixture placed in the work; in no other way can satisfactory results be obtained.

If it is intended to use reinforced concrete, great care must be taken that the plans are properly carried out. It is essential to place the reinforcement just as the design shows it. Numerous failures can be attributed to placing the steel in the forms without any attempt to make the reinforcement conform to the drawings. To do this means care on the part of the inspector, and no concrete should be permitted to be placed in the moulds until the steel skeleton has been carefully examined, accepted as placed, and passed on by the inspector. This is essential, especially when one considers the carelessness of carpenters, who make no attempt to remove from the forms pieces of wood, no matter how large. These, being enclosed in the concrete, naturally weaken the structure. It may or may not fall, but, usually, it will develop weak points.

In the concrete of a building which collapsed several years ago, were found, not only pieces of wood, but a large assortment of foreign matter, such as a pair of overalls which evidently had been placed in the form for safety, was forgotten, and eventually became part of the structure. It is such accidents, all due to carelessness, that have prevented a more rapid development of ferro-concrete construction in the United States.

Constant vigilance is necessary to insure good work and to obtain as nearly perfect results as possible. The entire work must be based on carefully drawn plans and specifications, and it is essential that they be followed faithfully.

E. G. WALKER,* ASSOC. M. AM. SOC. C. E. (by letter).—At first thought it might appear remarkable that so little progress relatively has Mr. Walker.

* Hull, England.

Mr. Walker. been made in the application of reinforced concrete to marine works in the United States; and in certain places in his paper, the author seems to attribute the present state of affairs to a conservatism of ideas on the subject utterly opposed to the general conception of the American progressive spirit. The writer is inclined to differ from this deduction, because he believes that in the United States as elsewhere the form of construction adopted for marine works is the result of economical evolution. A paper* on the piers of New York Harbor, recently presented before the Society, is an admirable illustration of this point, showing as it does in a very conclusive manner that special conditions evolve special types. A consideration of this paper leaves little doubt that the engineers of the Department of Docks and Ferries have arrived, by the judicious use of timber in some parts and reinforced concrete in others, at a type of pier of maximum economy under the conditions prevailing in New York Harbor. These conditions, however, are not comparable with those obtaining in certain teredo-infested waters on the coast of Great Britain, for example, where pitch pine piles will be dangerously eaten away in a few years and where even Australian hardwood piles have to be renewed at comparatively short intervals. Although there appear to have been a number of unfortunate experiences in the United States, there is no question that concrete and reinforced concrete can be used successfully for marine works, because engineers have had ample experience in the use of both materials; but there are many locations where it may be more economical to use timber, in spite of its shorter life and other disadvantages. At the present time there is a large quay under construction on the River Tyne, in England, forming the largest item in a $1 000 000 scheme, which will be built of timber throughout. In North America, where timber is so much more easily obtainable than in the United Kingdom, the writer thinks that it will be a long time before it is completely ousted by reinforced concrete.

In many cases in the United Kingdom the application of reinforced concrete to quay and jetty construction has been brought about by economical necessity rather than by ideas of progressiveness *per se*. Most of the waters of the British ports carry the teredo, and in many cases no timber but greenheart or jarrah will withstand its ravages. The Hennebique system of "ferro-concrete" construction, introduced into England by the late M. Mouchel, was first applied to sea-works at Southampton and, about the same time, to a jetty in the River Thames, at Purfleet. Both these works are referred to in detail in the paper. The success which attended them probably contributed very materially to the later popularity of reinforced concrete.

* "Modern Pier Construction in New York Harbor," by Charles W. Staniford, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 503.

In general, under average British conditions, greenheart construction is about as expensive as reinforced concrete, pitch pine being, of course, considerably cheaper than either. Hence, in the absence of special conditions (such, for example, as those at Port Talbot mentioned by the author), pitch pine may be used in places where the marine borers are not too active, whereas in other locations either hardwood or reinforced concrete may be used, according to the local balance of general economy. In connection with this, the proved low maintenance cost of concrete work is a most important factor, and it is significant that in most cases where concrete pile-work has been introduced, its use has been continued. Mr. Walker.

The remarkable resilience of concrete piles under driving has been a most important factor in the increase in their use, but the writer thinks that the particulars given by the author on page 1062 are scarcely applicable to a discussion on pile-work. It is reasonable to suppose that a hollow post of the dimensions given, reinforced with a large number of small rods, will be much more flexible than an ordinary pile, and such is the case. The resilience of piles is best exemplified by the treatment that they are able to stand in ordinary use, and particularly in driving. In the early days, the interposition of a special dolly between the pile and the monkey was considered essential, but since it has been found that properly arranged spiral winding is able to withstand all the punishment that the pile-driver can give it, the dolly is no longer used. This, in itself, is quite sufficient proof of the resilience of reinforced concrete piles; and there is plenty of other evidence.

A point which has arisen and which has been much discussed more particularly in connection with marine work in England is the subject of the relative advantages of pre-moulded members and members formed *in situ.* It has been contended that the former method is the better one by reason of its being more convenient, but the writer believes that, except where the special conditions of the case preclude it, it is desirable to build the structure as monolithic as possible. No matter how carefully the work is carried out, it is hardly possible to unite satisfactorily hardened concrete several weeks old with the new concrete that must be put in to form the joint; nor can one get in built-up work the strength obtained by the continuity of the steel from one member to another in monolithic work. The use of additional binding steel at the joints does not fully compensate for this. Advocates of the built-up system put forward the claim that monolithic work may be considerably weakened by the wrong disposition of the reinforcing bars, and that, therefore, it requires considerable and constant supervision; but surely the increased relative importance of the field-work in the case of pre-moulded construction entails quite as much detailed supervision on the job.

Mr. Douglas.

W. J. DOUGLAS,* M. AM. SOC. C. E. (by letter).—There have recently been completed, for the Port of Havana Docks Company, two large deep-water piers, built entirely of reinforced concrete. These are the San Francisco and Machina Piers, two- and four-story reinforced concrete structures with 85-ft. concrete pile foundations, in deep water and soft mud. Two additional piers are planned to be built at some future date.

The piers were designed and the construction directed by Barclay Parsons and Klapp, Consulting Engineers, of New York City. The contractors were MacArthur, Perks and Company, also of New York City.

Although these piers, with the large connecting warehouse along the shore, contain many features of interest to engineers, the present discussion will be confined to the pile foundation.

The piers are 164 ft. wide and from 660 to 680 ft. long, the distance between them being 262½ ft. The depth of the water varied from 20 to 40 ft., and the piles were driven into 15 to 20 ft. of soft mud which covered a firm sand and clay bottom.

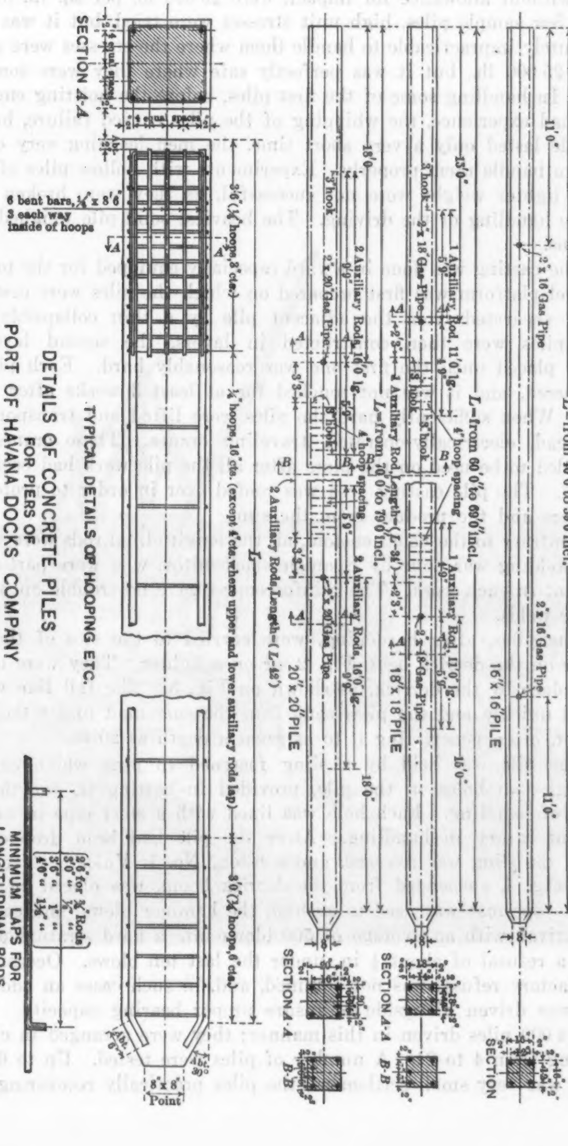
Prior to deciding on the use of concrete piles, careful consideration was given to various other types of foundations, but these piles were finally selected because, of all the permanent types of foundations, they were the cheapest. Marine borers are particularly destructive in these tropical waters, and this made it quite impossible to use wooden piles.

The length of the piles varied from 45 ft. inshore to 85 ft. in the outer 300 ft. The variation in length was made in 5-ft. increments. The piles were square in cross-section, the width of the sides being 16, 18, or 20 in. The piles were reinforced longitudinally with from 4 to 6-ft. plain round steel bars varying from ¾ to 1¼ in. in diameter. These main rods were hooped with ¼-in. square rods every 16 in. in the length of the pile, except where the rods of opposite faces lapped, and there the hoops were 4 in. from center to center. In the lower 3 ft. of each pile the hoops were 6 in. from center to center, and in the upper 3½ ft. there was a special rod reinforcement, basket-like in appearance, in order to resist the shock of the hammer. The details of the reinforcement are shown on Fig. 1.

For handling the piles, the longitudinal reinforcement was proportioned to the bending-moment stresses when the pile was supported horizontally from two points which were located so as to make the negative moments at the points of support about equal to the positive moment at the middle. Two methods of handling the piles were predetermined when they were designed. Fig. 2 shows one of these methods; the pile is about to be lowered into position by the derrick boat. The stresses resulting from handling the piles in this manner,

* New York City.

Mr. Douglas.



DETAILS OF CONCRETE PILES
FOR PIERS OF
PORT OF HAVANA DOCKS COMPANY
HAVANA, CUBA

FIG. 1.

Mr. Douglas. and without allowance for impact, were 20 000 lb. per sq. in. of steel. In a few sample piles, high unit stresses were tried, but it was found absolutely impracticable to handle them where the stresses were greater than 25 000 lb., but it was perfectly safe where they were somewhat less. In handling some of the first piles, before the hoisting engineers had had experience, the whipping of the piles caused failure, but this trouble lasted only a very short time, the men learning very quickly how to handle them properly. Experiments with hollow piles of somewhat lighter weight were not successful, as they were broken either in the handling or the driving. The heaviest solid pile weighed about 18 tons.

The casting was done in a yard especially equipped for the purpose. A level platform was first prepared on which the piles were cast, each being separated from the adjacent pile by a thin collapsible form. The piles were then constructed in layers, the second layer not being placed until the first one was reasonably hard. Each pile was numbered, and it was not touched for at least 3 weeks after it was cast. When sufficiently hard the piles were lifted and transported by overhead, electrically-operated, traveling cranes. These cranes were intended to be used on the piers after all the pile work had been completed. The pile casting area was roofed over in order to protect the laborers and the product from the sun.

Contrary to the usual custom, all the longitudinal rods were welded. The welding was done by Spanish blacksmiths, who were particularly efficient in such work. The reinforcement gave no trouble on account of the welds.

The piles, after hardening, were carried to the site of the work either on the derrick boat, Fig. 2, or on a lighter. They were lowered into place by the derrick, as shown on Fig. 3. The fall line was released quickly and the piles sank into the soft mud under their own weight, often penetrating it to as great a depth as 20 ft.

The pile was held by a sling fastened to pins which extended through two holes in the pile, provided in casting it, for the purposes of handling. Each hole was lined with a steel pipe in order to prevent injury in handling. After the pile had been dropped into place, the sling was released, and a 6-ton, No. 1, Vulcan, steam hammer, Fig. 4, suspended from the derrick boom, was placed on top of it. A rope mat was used to cushion the hammer blows, and each pile was driven with an average of 500 blows into a hard stratum until it gave a refusal of about ½ in. under the last ten blows. Occasionally, satisfactory refusal was not obtained, and, in such cases an additional pile was driven alongside to insure proper bearing capacity. There were 4 000 piles driven in this manner; they were arranged in clusters varying from 4 to 24. A number of piles were tested. Up to 60 tons there was very small settlement, the piles practically recovering when

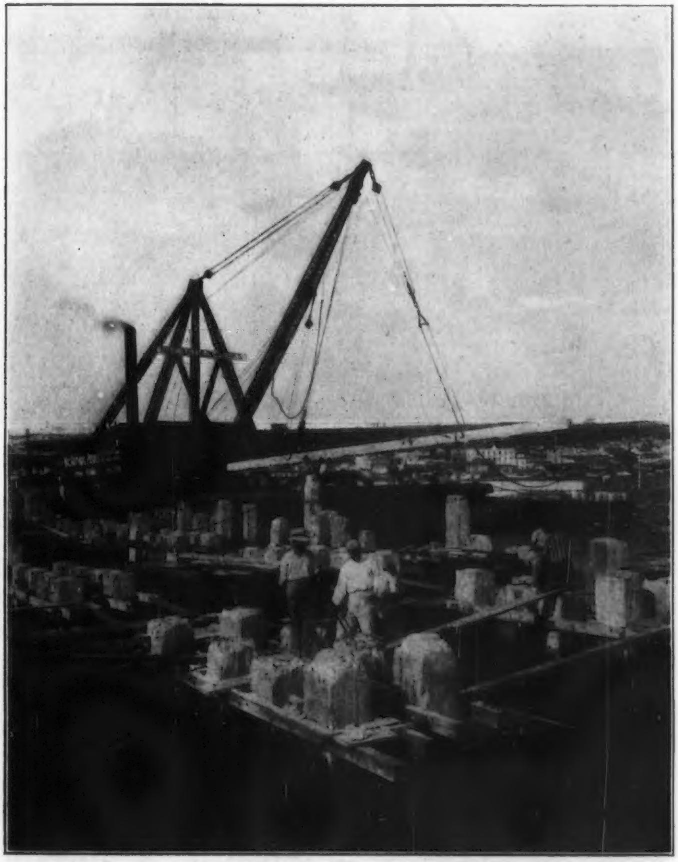


FIG. 2.—METHOD OF HANDLING THE PILES.

FIG. 3.—LOWERING A PILE INTO PLACE WITH THE DERRICK.

FIG. 4.—METHOD OF DRIVING THE BATTER PILES.

the test loads were removed. Each pile has a theoretic load of 40 tons, consisting of approximately 15 tons of dead load and 25 tons of live load. The tops of piles that could not be driven to the depths planned were cut off with a pneumatic chisel. Some trouble was caused by the concrete spawling off at the corners under driving. This was remedied by introducing ¼-in. square diagonal steel ties about 12 in. from center to center. Mr. Douglas.

It was also found that calcareous sand gave very much stronger pile heads than ordinary sand or stone dust. The stone dust was particularly unsatisfactory, resulting in a great many breakages of the pile heads under driving.

Piles were driven at an average rate of about 12 a day. Only two lines of batter piles at the outer end of each pier were used, notwithstanding the great depth of water. For the lateral stiffness of the piers, the designer depended almost entirely on the resistance of the floor as a cantilever anchored inshore. This assumption seems to be warranted fully by the stiffness of the piers. As the concrete floors are heavily reinforced, it was assumed that any shock of boats against the pier would be transferred from the outer end, through the concrete and steel reinforcement, to a point where the piles were sufficiently short to insure ample stiffness. Fig. 4 shows the method of driving the batter piles.

Previous to designing and constructing these piers, an examination was made of some concrete wharves in which the longitudinal reinforcement above water had caused the steel to rust and spoil the concrete. Quite a number of rods were exposed to sea action. This was due to the fact that the steel had not been spaced carefully. Extreme care was taken in this Havana work to keep the longitudinal steel well covered with concrete. In other cases it had been observed that, where tie wires had been left protruding from the faces of the piles, rusting had taken place and, of course, had worked in until there was disintegration of the concrete. The piles in these piers have been examined from time to time and have been found to be in perfectly good condition. At present, of course, they are fully covered with marine growth below the water line.

HARRISON S. TAFT,* ESQ. (by letter).—The discussion on this paper may be divided into two classes; namely, those by men who have made a thorough study and investigation of the subject, covered by visits to foreign ports; and by men whose knowledge and experience has been confined to the few years during which concrete docks have been building in the United States. Mr. Taft.

The writer regrets that he has not been able to include in the discussion one or two communications he has received from foreign dock engineers, giving the results of their experience with concrete

* Seattle, Wash.

Mr. Taft. when subjected to sea water action, because in at least one instance, these results have been radically different from those at some American ports. Whereas, what few failures have taken place in the use of concrete in sea water structures in America seem to have been between tides, it appears that at Southampton, England, "below high water of neap tides the reinforced concrete is absolutely perfect, except in a few places where evidently it has been damaged by blows * * * the rusting action all taking place above high tide level". The writer's informant states that electrolysis has been the cause of this rusting, and that, in building concrete docks, it is of vast importance that "the electric mains should be carefully insulated so that the current cannot pass through the reinforcement".

As for any attempt at expanding the contents of the paper, the writer feels that the ground, in general, was covered so thoroughly that no further discussion is needed at present. He admits that the whole subject of the use of cement in sea water structures is one in the actual handling of which a large amount of knowledge has yet to be acquired by concrete engineers, and an equal amount of training given to concrete foremen and their crews, before they can become past masters in the art. For, from what does the foreign engineer get his confidence in concrete structures exposed to sea water conditions, but from actual experience and past successes. On the other hand, whence comes the skepticism of American engineers on the same subject, but from lack of thorough knowledge of the art and failures resulting therefrom.

Mr. Stern seems to feel that the writer did not dwell sufficiently on the failure side of the question, and overlooked a case, which he cites, in the lower part of New York City. It appears to the writer that in Part III of the paper sufficient note was made of a number of bad failures on both the Atlantic and Pacific Coasts to show their extent, without citing each and every case that ever happened. The writer most certainly did not try to bring out the successful uses of concrete subject to sea water action more prominently than the other side of the discussion; but endeavored to treat the subject matter of Part III on an impartial basis, and regrets that answers were not forthcoming in explanation of some of the failures in the use of concrete mentioned therein.

As Mr. Stern pointed out, cement chemists and manufacturers must give the concrete engineer a cement suitable for sea water structures, if the art of building concrete docks is to progress; but it is of equal importance to have a suitable sand, gravel, or stone, and of still more importance that the contractors shall handle the material in such a way as to obtain an impermeable concrete, because, in the final analysis, and as Mr. Coombs has brought out, pre-moulded concrete, of a rich mixture, made of suitable materials is the prime requisite

for the most successful use of concrete when subjected to sea water action. In no case should the salt water come in contact with green concrete. Mr. Taft.

In stating that in using pre-moulded concrete the jointing together of the different members of the structure into a monolithic mass is a difficult accomplishment, and that the existence of joints below the water is of a serious nature, Mr. Walker utters a truism. There is no doubt that a true monolithic structure is desirable, if such a mode of construction can be adopted economically. On the other hand, has the pre-moulded system reached its final and ultimate development, and do not pre-moulded concrete piles with their heads embedded in the deck members, the latter poured *en masse, in situ,* offer a remedy for the objectionable features, mentioned by Mr. Walker, in such a system of construction?

The writer doubts very much that the art of concrete construction under sea water conditions has yet reached its ultimate development. In fact, he knows it has not, or else concrete engineers would be at a standstill, and such a condition is absolutely contrary to American engineering. During the past year or so the writer has been working on this very problem in order to overcome the objectionable features in the pre-moulded system, which he recognized some time ago, namely, the joints in concrete exposed to sea water action where the steel reinforcement passes through the joint. If such a thing can be obliterated—and it is possible to do so—does not such an objection to a pre-moulded structure become a negative quantity?

In reviewing the criticism by Mr. Davis, it would appear that he and the writer had gone over the same road, in their study and investigation of the subject, as there seems to be hardly a thought in Mr. Davis' most comprehensive statements which does not coincide with the writer's ideas and with the ideas of a large number of leading dock engineers of foreign countries. Mr. Davis' discussion is full of most valuable suggestions, relating to the construction side of the problem, as well as the actual handling of the concrete material of dock work, and can be studied with great advantage by all engineers engaged in such work.

Through all the discussions the writer has noted references to questions as to the commercial life of temporary *versus* permanent structures. As Mr. Barney points out, and as stated previously by the writer,* the one great question in all such discussions is that of the "financial and commercial life"—a vital factor only too often ignored. At some future time the writer trusts that he will have the privilege of laying before this Society the results of his studies of

* In his discussion of the paper by Charles W. Staniford, M. Am. Soc. C. E., entitled "Modern Pier Construction in New York Harbor," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 525.

Mr. Taft. dock finance, wherein, with charts already prepared, the relative position of permanent and temporary dock structures is clearly portrayed. Briefly stated, the problem is this:

If, during the life of a wooden structure, it can earn its depreciation charges, fire insurance, operating expenses, interest on investment (supposedly a bond issue, as with most public improvements), and pay off the bonds, then perhaps such a structure is of advantage, except with reference to the general fire hazards to surrounding property; but, if the wooden structure cannot do this, and the bond issue is for a longer period than the life of the dock, a permanent structure is more advantageous than a temporary one.

In closing, the writer desires to thank the members of the Society for the courtesy shown him in the generous treatment of his paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1328

A METHOD OF DETERMINING STORM-WATER RUN-OFF.*

By Charles B. Buerger, M. Am. Soc. C. E.

With Discussion by Messrs. R. C. Strachan, Kenneth Allen, W. W. Horner, Charles W. Sherman, Robert E. Horton, C. E. Grunsky, Samuel D. Bleich, Charles E. Gregory, Samuel A. Greeley, W. E. Fuller, George W. Fuller, and Charles B. Buerger.

Synopsis.

The rational method of determining storm-water run-off, as introduced by Emil Kuichling, M. Am. Soc. C. E., and modified in various particulars by designers in the United States and abroad, has usually given satisfactory results. It is not very widely used, however, and the formula methods, of which the Bürkli-Ziegler and the McMath are the most popular, are most generally used, in spite of the common realization of the fact that the results given by them lack consistency, and are very erratic and unreliable. The popularity of these insufficient formulas, notwithstanding their drawbacks, indicates that a formula which will give results reasonably close to those obtained by the rational method will be acceptable for service, even though of somewhat complicated form.

The writer has developed a formula for storm-water run-off of the form

$$q + N q^{\frac{4}{5}} = P$$

in which q is the run-off, in cubic feet per second per acre, and N and P are functions of the variable elements of topography, rainfall,

* Presented at the meeting of October 7th, 1914.

etc. Diagrams are given which afford a ready means of obtaining results from this formula.

The writer compares the results obtained by this formula with the gaugings in Rochester by Mr. Kuichling, the Birmingham gaugings by Mr. Lloyd Davies, the Evanston gaugings by C. C. Saner, Assoc. M. Am. Soc. C. E., the New York gaugings by Rudolph Hering, M. Am. Soc. C. E., the St. Louis observations by Robert E. McMath, M. Am. Soc. C. E., the Chicago observations by C. D. Hill, M. Am. Soc. C. E., and others; and notes the variations and discrepancies between observations and calculated results by the new formula. He considers that the results given by his formula are reasonably near to those of the gaugings, and that they may be considered an acceptable equivalent to those determined by the cut-and-try rational method.

Introductory.

At the present time much thought is being devoted to the question of storm-water drainage. Many large cities, such as New York, Chicago, St. Louis, Cincinnati, and others, are studying their storm-water systems, and in some cases are rebuilding, where time has proved earlier designs to be inadequate.

Much of the drainage work done throughout the United States, even for large cities, does not have the benefit of such study, but is based on formulas entirely unsuited to the particular local conditions.

On this subject, it has been remarked, with some appearance of justice, that a refinement of methods in the calculation of storm-water run-off is not warranted by any experimental data now available; and it is implied that present methods are good enough when bearing in mind the meagerness and inconsistency of field determinations available to check these methods. It does not appear, however, that the best possible use has been made of existing gaugings.

Existing Methods.

Broadly speaking, two methods of attacking this problem are in customary use: The first is the so-called rational method, with its manifold modifications and variations; the second is the formula method. The rational method was publicly introduced by Mr. Kuichling in 1889.* Mr. August Frühling has given the most complete

* *Transactions*, Am. Soc. C. E., Vol. XX, p. 1.

and satisfactory exposition of this method.* Modified rational methods have been elucidated by Mr. E. E. Wallington Butt,† Mr. Carl H. Nordell,‡ Mr. Maximilian Vicari,§ (the Hauff method), C. E. Grunsky, M. Am. Soc. C. E.,|| Mr. Jean Balcomb,** W. W. Horner, Assoc. M. Am. Soc. C. E.,†† and Messrs. R. O. Wynne-Roberts and T. Brockman.‡‡

The various applications of the rational system differ much in detail of use, but they all follow the system of reproducing, by cut-and-try methods, the existing conditions, to the point of finding corresponding sewer grades, sizes, and assumed rainfall curve and rain absorption or retention.

It is almost everywhere conceded that the rational system, in any of its forms, if used with proper discretion, will give very satisfactory results. It is worth noting, however, that it has not received the widest use, and that formula methods are much more popular. For this there is the reason that the rational system is relatively laborious, and requires, in addition, a material exercise of judgment. Not but that frequent practice will rapidly teach facility of application; but the formula systems have always carried, and probably always will carry, the popular support, because of their convenience and mechanical system of use.

Many run-off formulas have been proposed, among which may be listed:

Hawksley (or Bazalgette):

$$\log. d = \frac{3 \log. A + \log. N + 6.8}{10} \quad \text{(1)}$$

Where d = diameter of sewer, in inches;

N = length of sewer per foot of drop;

and A = area, in acres.

Bürkli-Ziegler:

$$q = C\,R\,\frac{S^{0.25}}{A^{0.25}} \quad \text{(2)}$$

* "Handbuch der Ingenieurwissenschaft: Der Wasserbau;" Die Entwässerung der Städte, Leipzig, 1910 (William Engelmann).

† *The Surveyor*, 1907, Vol. XXXII, p. 132.

‡ *Proceedings*, Municipal Engineers of the City of New York, 1909, p. 6, "An Additive Method of Run-off Determination for Storm-Water Sewers."

§ *Gesundheits-Ingenieur*, 1909, Vol. 32, p. 569.

|| *Transactions*, Am. Soc. C. E., 1909, Vol. LXI, p. 496, "The Sewer System of San Francisco, and a Solution of the Storm-water Flow Problem."

** *Proceedings*, Western Society of Engineers, 1910, Vol. XV, p. 699, "The Design of Storm-water Drains in a Modern Sewer System."

†† *Engineering News*, 1910, Vol. 64, p. 326, "Modern Procedure in District Sewer Design."

‡‡ *Canadian Engineer*, 1913, Vol. 24, p. 437.

where q = run-off, in cubic feet per second per acre;

C = an empirical constant;

S = slope, in feet per 1 000 ft.;

A = area, in acres;

and R = rainfall, in inches per hour.

Adams:

$$q = C\, R^{0.83} \frac{S^{0.083}}{A^{0.167}} \dots\dots\dots\dots (3)$$

McMath:

$$q = C\, R \frac{S^{0.20}}{A^{0.20}} \dots\dots\dots\dots (4)$$

New York diagrams (Hering):

$$q = C\, R \frac{S^{0.27}}{A^{0.167}} \dots\dots\dots\dots (5)$$

W. C. Parmley, M. Am. Soc. C. E.:

$$q = C\, R \frac{S^{0.25}}{A^{0.167}} \dots\dots\dots\dots (6)$$

C. E. Gregory, Assoc. M. Am. Soc. C. E.:

$$q = 2.8 \frac{S^{0.187}}{A^{0.14}} \dots\dots\dots\dots (7)$$

$$q = \frac{\dfrac{105\, C}{L}}{84 \sqrt[5]{A\, S^2}} + 25 \dots\dots\dots\dots (8)$$

When L = greatest length of sewer, in feet.

All these formulas have this feature in common, that they were devised to represent the conditions at some particular locality, and within certain limiting ranges of slope, area, length, rainfall, and imperviousness. There is no reason to doubt that, for the conditions they were formed to meet, they gave results approximating the facts. Some of them, notably the Bürkli-Ziegler and the McMath, have found a wider application than is reasonably warranted.

An engineer can use a formula with some confidence if sufficient experience is available to assure him that his constants are the proper ones and that he is applying it within its allowable range. In a city, say, like New York, where any formula has been successfully used (and that implies that resulting construction has been neither too large nor too small), and where the conditions of use being con-

sidered are within the range of previous practice, the engineer can be satisfied to design additional storm sewers on the same basis; but, unfortunately, it is only occasionally that it can be determined whether or not the design has been a proper one, from an engineering standpoint.

A great many of the designs for storm-water sewers do not come in this class. They are made for localities where there is little precedent to serve as a guide. The attempt to use a formula of the ordinary design has here at times resulted disastrously. The data of topography and rainfall, of course, are at hand, but, in a new territory, the proper selection of the constants is a matter of much difficulty. For instance, in the exponential formula (McMath type), Cleveland has used a value of $R = 4.0$; New York a value of 2.75; and Chicago a value of 1.0. The rainfall in these cities does not differ materially, and in no way accounts for this variation in the assumed R.

It is not necessary at this late date again to point out the limitations and discrepancies of any of the various formulas used. These are generally realized. In spite of this, it is probable that the great bulk of storm-water drain design is still being executed on the basis of these discredited methods. It is not unusual to read an account of a storm-water drainage system in which all details of construction have been studied most carefully, and where materials, slopes, grades, strength, and other factors are fully worked out. The basis of design —the determination of the needed sizes—will be dismissed in a short paragraph stating that the Bürkli-Ziegler or McMath formula was used, with certain constants; and it is likely that, for a mountainous, rocky district of small size, the same constants may be used as for a flat beach town of large extent. It seems to be established that the general preference is for the use of some kind of a formula, and that a reasonably correct and comprehensive one should be an acceptable substitute for the ordinary rough approximations.

There is a need for a formula of general application which can be used in a new locality with some reasonable assurance that it will be fairly appropriate, will be applicable for large areas or small, flat or steep slopes, and heavy or light rainfalls. The results given by such a formula must be compared fairly with the best of experimental information of storm-flow gaugings, and appear to be an envelope

of the bulk of the gauging results, rather than a mean line through them, in order to show a fair margin of safety.

It is not to be expected that such a formula can be a very simple expression; but this sort of complication is not a heavy deterrent to general use, if the results command confidence, as is evident by the general use of the Kutter formula.

The writer has developed such an empirical formula for storm-water run-off, and it is presented herein, together with a detailed comparison with all published gaugings available to him.

The Writer's Run-Off Formula.

q = the run-off, in cubic feet per second per acre;

L = length of longest sewer run, in feet;

C = a coefficient, the ratio of rainfall eventually reaching the sewers to the total rainfall;

A = area, in acres;

S = average slope of longest sewer, L, in feet per 1 000 ft.;

m = average slope of ground surfaces to sewer inlets, in feet per 1 000 ft.;

K = inches of rainfall per hour, lasting for a period of 1 hour, which sewers are to discharge.

$$N = \frac{L}{79 \sqrt[5]{C A S^2}\left(24 + \frac{16}{\sqrt{m}}\right)} \qquad (9)$$

$$P = \frac{3.33 \sqrt{m}\, K C}{\sqrt{m} + 0.87} \qquad (10)$$

$$q + N q^{\frac{4}{5}} = P \qquad (11)$$

Comparison with Run-Off Data.

The validity of a formula of this type rests only on its corroboration by the best of available data. A full coincidence, of course, cannot be expected. The data are scanty, not consistent in themselves, and not always such as to be entirely credible; but it is necessary that the formula be compared with the information we have for all known widely varying conditions.

KUICHLING'S ROCHESTER GAUGINGS.*

Mr. Kuichling's data are fairly full, but very far from being complete. It is necessary at times to estimate certain values as the probable ones, and where that is done a note to that effect is made.

District IV.—Size, about 4 800 by 1 200 ft.; population, 32 per acre; area, 128.67 acres; average street grades, 1 in 130; sewer grades, 1 in 50 to 1 in 630; time to flow through longest sewer, 18 min.; time to reach sewer, 8 min.

$A = 128.67$;

$L = 5\,000$ ft. (estimated);

$m = 7.7$;

$S = 2$;

$C = 0.34$;

I, for time of concentration (26 min.), from Kuichling's Plate I, $= 1.55$;

K, from Kuichling's Plate I, $= 0.87$.

District I.—Area irregular, 356.94 acres; about one-half of the area has a population of 35 per acre; remainder, agricultural and thinly settled; grade of sewered streets, 1 in 150; sewer grades, 1 in 47 to 1 in 910; time to flow through sewers, 34 min.; time to reach sewers, 10 min.

$L = 9\,300$ ft. (estimated);

$m = 6.7$;

$S = 1.4$ (estimated);

$C = 0.25$;

I (for 44 min.) $= 1.0$;

K (as before) $= 0.87$.

District X.—Long, narrow strip; area, 25.12 acres; population, 40 per acre; surface grades, 1 in 172; sewer grades, 1 in 70 to 1 in 330; time to flow through sewers, 10 min.; time to reach sewers, 6 min.

$L = 2\,000$ ft. (estimated);

$m = 5.8$;

$S = 4.0$ (estimated);

$C = 0.39$;

I (for 16 min.) $= 1.78$;

K (as before) $= 0.87$.

* *Transactions*, Am. Soc. C. E., 1889, Vol. XX, p. 1.

District IX.—Shape not known; area, 132.96 acres; population, 36 per acre; street grades, 1 in 151; sewer grades, 1 in 54 to 1 in 400; time to flow through sewers, 15 min.; time to reach sewers, 8 min.

$L = 4\,000$ ft. (estimated);
$m = 6.6$;
$S = 3.0$ (estimated);
$C = 0.37$;
I (for 23 min.) $= 1.6$;
K (as before) $= 0.87$.

District XVII.—Shape unknown; area, 92.27 acres; population, 35 per acre; street grades, 1 in 240; sewer grades, 1 in 100 to 1 in 350; time to flow through sewers, 16 min.; time to reach sewers, 8 min.

$L = 3\,500$ (estimated);
$m = 4$;
$S = 3.0$ (estimated);
$C = 0.36$ (estimated);
I (for 24 min.) $= 1.6$;
K (as before) $= 0.87$.

Table 1 gives the comparative results of gaugings and computed run-offs.

TABLE 1.

District No.	Highest gauging. Q (cubic feet per second).	Kuichling. Rational method.	Buerger formula.
IV	71.3	67.5	51.0
I	77.01	92.5	78.5
X	21.04	17.4	16.0
IX	46.0	78.4	71.0
XVII	28.45	53.0	46.5

The comparison in Table 1, of highest gaugings and calculations based on the assumed rainfall curve, is not entirely responsive. A detailed comparison of the special storms is given in Table 2.

For this purpose, it is assumed that K can be determined by the relationship,

$$I = \frac{80\,K}{20 + t} \quad \text{.......................(12)}$$

where I is the intensity for a duration of t minutes,

$$\text{or } K = I(0.25 + 0.0125t) \quad \text{...............(13)}$$

TABLE 2.—KUICHLING'S GAUGINGS.

Date, 1888.	Maximum intensity of rainfall.	Duration of rain at maximum intensity.	K, Estimated.	District I.			District IV.			District X.			District IX.			District XVII.			Percentage, average by storm.
				Q		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	Q		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	Q		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	Q		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	Q		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	
				Buerger.	Gauging.		Buerger.	Gauging.		Buerger.	Gauging.		Buerger.	Gauging.		Buerger.	Gauging.		
May 9th....	1.315	35	0.90	78.5	77.01	102	......	......	......	......	......	......	......	......	......	......	......	......	......
	1.00	35	0.69	......	......	......	41.2	33.7*	......	......	......	......	......	......	......	......	......	......	......
	0.75	35	0.52	......	......	......	......	......	......	9.3	9.81*	......	40.0	28.99*	......	26.6	17.97*	......	......
May 26th...	1.00	13	0.41	28.5	30.76	98	20.6	33.3	62	6.8	7.94	86	30.5	24.86	123	18.5	10.84	170	106.8
June 24th..	1.55	30	0.97	85.5	40.72	210	......	......	......	......	......	......	......	......	......	......	......	......	210.0
June 24th..	2.62	20	1.31	117.0	58.82	199	77.0	71.26	108	24 0	21.04*†	......	106.0	46.0*†	......	70.0	28.45*†	......	153.5
June 28th..	0.80	20	0.40	28.5	40.72	70	20.6	29.54	70	6.8	7.09	96	29.2	37.5	77	18.5	27.66	67	76.0
July 11th...	0.76	15	0.33	25.0	19.93	125	16.7	15.44	108	5.5	8.01	69	22.6	22.09	104	15.6	13.57	115	104.2
Aug. 4th....	1.00	12	0.40	28.5	16.54	169	20.6	12.75	163	......	......	......	29.2	19.88	147	18.5	12.71*†	145	156.0
Aug. 16th...	1.616	15	0.71	64.0	27.15	236	41.2	25.93	159	12.6	10.01	126	56.0	38.23	146	37.0	28.45	......	166.8
Aug. 17th...	1.333	10	0.5	43.0	25.80	167	27.0	14.94	181	8.0	6.17	130	37.1	21.12	176	25.0	10.94	228	176.4
Aug. 26th...	2.50	14	1.01	89.0	35.33*	......	59.1	39.28	150	18.4	21.04*†	......	77.0	46.0*	......	55.3	28.45*†	......	150.0
Sept. 16th..	0.47	50	0.41	28.5	33.25	84	20.6	23.12	89	......	......	......	......	......	......	18.5	16.14	115	96.0

* Intensity roughly estimated.

† Sewers ran under head ; figures given are for maximum discharge without head, or previous to surcharge.

Lloyd Davies' Birmingham Gaugings.*

District I.—Moseley Street.—Shape unknown; area, 312.5 acres; population, 125 per acre; ground slope, 1 in 60; sewer grade, 1 in 85; length, longest line of sewers, 9 565 ft.; time to flow through sewers, 16.75 min.; time to reach sewers, 1.25 min.

$m = 16.7$;
$S = 11.8$;
$C = 1.00$ (estimated);
I (for 18 min.) $= 1.1$.

District II.—Charlotte Road.—Shape unknown; area, 232 acres; population, 4 000; number of buildings, 500; street and road area, 10%; impermeable, 18%, increasing to 42% as the permeable area becomes saturated; length of longest line of sewers, 4 454 ft.; time to flow through sewers, 9.4 min.; time to reach sewers, 2.6 min.; sewer grades, 1 in 55.

$m = 20$ (estimated);
$S = 18.2$;
$C = 0.18$ (estimated);
I (for 12 min.) $= 1.8$.

Table 3 gives a comparison of these gaugings and calculated results, K being determined in the same way as for the Kuichling gaugings.

TABLE 3.

Date, 1904.	District I.				District II.				Average percentage for storm.
	K.	Q.		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	K.	Q.		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	
		Buerger.	Gauging.			Buerger.	Gauging.		
Jan. 10th..	0.155	62.5	95.0	66.0	......	....	....	....	66.0
May 27th..	0.345	172.0	248.0	69.5	0.27	32.5	42.0	77.5	73.5
July 26th..	0.41	203.0	261.0	77.9	0.44	51.0	63.2	80.9	79.5
Aug. 5th..	0.22	93.7	60.1	156.0	0.27	32.5	29.6	110.0	133.5

* *Minutes of Proceedings*, Inst. C. E., 1906, Vol. CLXIV, p. 41.

SANER'S EVANSTON GAUGINGS.*

TABLE 4.—DAVIS STREET SEWER.

Date, 1912.	Time of concentration, in minutes.	Rate of rainfall, in inches.	Area, in acres.	L, in feet, estimated.	C.	S, estimated.	m, estimated.
June 4th...	15	0.60	141	5 900	0.20	5	5
July 3d....	10	2.10	100	5 000	0.20	8	8
July 21st...	45	0.15	420	10 200	0.20	2.5	3
May 28th..	45	0.32	420	10 200	0.20	2.5	3
Aug. 20th..	10	0.78	100	5 000	0.20	8	8

Table 5 gives a comparison of calculated and observed run-off.

TABLE 5.

Date, 1912.	K.	q, in cubic feet per second per acre.	Q, in cubic feet per second.	Q, by gauging.	Ratio : Q-Buerger / Q-Gauging
June 4th....................	0.25	0.075	10.6	11.8	90.0
July 3d	0.79	0.24	24.0	27.0	89.0
July 21st....................	0.12	.....	....	10.0	....
May 28th....................	0.26	0.03	12.6	30.0	42.0
Aug. 20th....................	0.27	0.08	8.0	5.6	143.0

HERING'S NEW YORK GAUGINGS.†

Sixth Avenue Sewer.—Area = 221 acres; population = 171 per acre; time to flow through sewers = 15 min.

$m = 7$ per 1 000;

$S = 3.5$ (estimated);

$L = 4\,800$ ft.;

Surface, 96.64 acres roof area $= 43.5\%$;

103.32 " paving $= 46.5\%$;

21.76 " grass $= 10\%$;

$C = 0.76$ (estimated).

* *Journal*, Western Society of Engrs., 1913, Vol. XVIII, p. 698.

† *Transactions*, Am. Soc. C. E., 1907, Vol. LVIII, p. 468.

Table 6 contains the data of the heavier storms used for comparison.

TABLE 6.

No.	Date.	Maximum rainfall, in inches.	Duration of maximum rainfall, in minutes.
2.....	Dec. 28th, 1887	0.149	4
18.....	June 26th, 1888	0.583	13
19.....	June 28th, 1888	0.118	4
21.....	July 19th, 1888	0.394	10
23.....	Aug. 4th, 1888	0.590	10
25.....	Aug. 21st, 1888	0.396	10
26.....	Sept. 11th, 1888	0.200	10
27.....	Sept. 13th, 1888	0.299	7
28.....	Sept. 19th, 1888	0.410	16
35.....	Nov. 8th, 1888	0.187	5

Table 7 gives a comparison of computed and gauged run-off.

TABLE 7.

No.	K.	Q.		Ratio:
		Computed (C. B.B.)	Gauged.	
2.......	0.67	199	64.3	310.
18.......	1.11	354	200*	177.
19.......	0.53	166	180.9*	92.2
21.......	0.88	277	147.7	187.0
23.......	1.33	398	200*	200.
25.......	0.88	277	200*	188.5
26.......	0.45	143	138.8	103.0
27.......	0.87	277	146.6	189.0
28.......	0.69	199	157.4	127.0
35.......	0.70	199	200	100.0

* The original table does not agree in various columns.

McMath's St. Louis Observations.*

The information in Mr. McMath's paper is far from being accurate enough to receive much weight. The notes as to overcharge of sewers are not more than observations. It is most difficult to estimate what is intended by the expression "often overcharged." The writer has assumed in the following comparison that a value $K = 1$, corresponding to full discharge in a period of about $1\frac{1}{2}$ years, meets this condition. In all cases, $C = 0.75$, and $m = 15$.

Table 8 gives a comparison of Mr. McMath's records and the calculated run-off.

* *Transactions*, Am. Soc. C. E., 1887, Vol. XVI, p. 179.

TABLE 8.

Location.	A.	L, estimated.	S, estimated.	Required Q McMath.	Required Q Buerger.	Actual Capacity, Calculated. Q.	Actual Capacity, Calculated. Remarks.
Compton Avenue (2)..	284	6 800	8.	330	425	253	Often overcharged.
Camp Spring (1)......	55	3 000	8.7	87	99	100	" " *
" " (6)......	516	9 100	13.	524	770	733	" " *
" " (2)......	49	2 800	11.	80	93	86	" " *
Biddle Street (2)......	82	3 600	4.	120	122	60	" "
" " (5)......	441	8 400	6.	462	619	496	" " *
" " (6)......	458	8 600	6.	475	641	605	" " *
" " (7)......	657	10 200	5.5	640	850	1 165	" " *
Northwestern (1)......	113	4 300	9.4	154	171	102	" "
" (2)......	138	4 700	10.	183	221	178	" "
Cass Avenue (1)......	61	3 100	2.3	96	92	49	" "
" " (3)......	94	3 900	6.	134	141	151	" " *
" " (5)......	190	5 500	5.5	237	284	180	" "
Benton Street (2)......	155	5 000	7.	200	248	163	" "
Grand Avenue (3)......	138	4 700	6.	182	220	122	" "
" " (4)......	198	5 600	5.5	246	307	138	" "
" " (5)......	236	6 100	5.	283	341	266	" "
" " (6)......	271	6 600	5.	312	392	325	" "
" " (7)......	304	7 000	5.	348	440	256	" "
Elliot Avenue (1)......	31	2 200	15.	55	56	52	" "
Broadway Branch (1).	33	2 300	7.8	58	59	40	" "
" " (2).	46	2 700	9.	77	83	43	" "
" " (3).	51	2 900	10.	83	89	55	" "
" " (4).	66	3 200	11.	102	116	90	" "
Salisbury Street (1)...	49	2 800	19.	81	88	60	" "
" " (2)...	91	3 800	12.	131	159	98	" "
Ferry Street (4).......	338	7 400	11.	373	506	274	" "
Trudeau Street (1)....	120	4 400	16.	163	204	112	" "
Arsenal Street Branch (1)..	36	2 400	13.6	62	65	50	" "
Arsenal Street Branch (2)..	49	2 800	12.	82	88	48	" "

* Mr. McMath attributes overcharge to local conditions, rather than to insufficient capacity.

Hill's Chicago Observations.*

Mr. Hill's observations resemble Mr. McMath's in that the record of results is vague, and consists only of a statement of the adequacy or inadequacy of the sewer.

In considering the information given, allowance must be made for a later statement by Mr. Hill,† from which the following is quoted:

"Since the paper under discussion was written, there have been a number of severe rainstorms, that have again demonstrated the inadequacy of many of Chicago's sewers, and have severely tested others that were previously above suspicion. To some extent the opinions of the writer have been modified. On the evening of May 24th there was a downpour aggregating nearly two inches, with a maximum intensity of three inches per hour during a period of fifteen

* *Journal*, Western Society of Engrs., 1902, Vol. VII, p. 425.

† *Loc. cit.*, p. 442.

minutes. The result of this storm was that nearly every deep basement in the city was flooded."

In Table 9, for the purpose of comparison, the run-off as calculated by the Hill method, as outlined in his paper, is given in the column so marked. The coefficient, C, is estimated from the population and the relatively impervious areas noted. The sewer grade, S, is taken as 0.50, and the street grades, m, as 2.0 per 1 000. K is taken as 1 in.

Water-shed Flood Run-off.

Table 10 contains data relating to the maximum flood discharge of streams in the eastern section of the United States, for water-sheds up to 100 sq. miles in area.*

The flood discharges in Table 10 are plotted on the diagram, Fig. 1, by their numbers, together with two curves drawn according to the proposed run-off formulas for values of K of 1.0 and 1.5, respectively.

For comparison, the same assumptions are made throughout.

$L = 300\ \sqrt{A}$, in acres;

$C = 0.15$;

$m = 15$;

$S = 10$;

$K = 1.0$ and 1.50.

Comparison with Other Computations.

Walworth Sewer, Cleveland, Ohio.†—The data and computations used by Mr. W. C. Parmley are taken from Table 2, page 347, of his paper on this sewer. From Fig. 1, on page 351, the highest rainfall, lasting 1 hour, and corresponding to $R = 4$, is taken as $K = 1.1$. From the rainfall records, the indications are that this sewer will never be more than filled. This, of course, is much more liberal than ordinary design, which permits of periodic flooding.

Table 11 gives comparative results of the methods used.

St. Louis Sewers.

Table 12, giving comparative results of the rational method by Mr. W. W. Horner, and the proposed formula, is made up from an article by Mr. Horner.‡

* These are from data by Mr. Kuichling in the New York Barge Canal Report, 1901, p. 852 *et seq.*, and *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 650 *et seq.*

† *Transactions*, Am. Soc. C. E., 1905, Vol. LV, p. 341.

‡ "Modern Procedure in District Sewer Design," by W. W. Horner, *Engineering News*, 1910, Vol. LXIV, p. 328.

TABLE 9.

	Location of Sewers.	Area, in acres.	Population per acre.	Impervious area.	C (Estimated.)	Run-off, in Cubic Feet per Second per Acre.		Sewer capacity, in cubic feet per second per acre.	Observation.
						Hill.	Buerger.		
A1	West Adams St., West of River	266	70	0.90	0.54	0.20	0.35	0.20	Adequate.
A2	Armitage Ave., West of River	3 280	40	0.30	0.34	0.07	0.11	0.08	Adequate as yet.
A3	Adams St., East of River	30	1 500	1.00	1.00	0.86	0.36	1.56	Few complaints.
A4	Addison St., East of Robey St.	300	30	0.30	0.26	0.19	0.17	0.147	Adequate.
A5	Augusta St., West of River	196	120	0.90	0.75	0.21	0.56	0.26	
B1	Bryn Mawr, West of Lake Michigan	275	10	0.20	0.12	0.083	0.065	0.12	Adequate.
B2	Belmont Ave., West of Lake View Ave.	85	60	0.50	0.50	0.264	0.44	0.20	Basements flooded.
C1	North Clark St., North of River	400	90	0.80	0.64	0.18	0.51	0.22	Adequate.
C2	Clybourne Place, East of River	180	90	0.40	0.31	0.135	0.23	0.16	"
C3	Chicago Ave., West of River	48	100	0.90	0.70	0.76	0.98	0.33	"
C4	Chicago Ave., East of River	108	120	0.90	0.75	0.264	0.92	0.25	"
D1	Dearborn St., Randolph to Madison	1.75	1 500	1.00	1.00	1.74	1.81	1.71	Few complaints.
D2	Division St., West of River	367	100	0.90	0.70	0.18	0.63	0.14	Doubtful.
D3	Division St., East of River	50	10	0.40	0.12	0.31	0.11	0.33	Adequate.
D4	Diversey Boulevard, East of River	675	30	0.40	0.26	0.10	0.17	0.113	Doubtful.
E1	Erie St., West of River	33	100	0.90	0.70	0.84	1.05	0.50	Adequate.
F1	S. 45th Ave., North of River	3 800	40	0.30	0.31	0.07	0.12	0.76	Undeveloped.
F2	Foster Ave., West of Lake Michigan	265	10	0.20	0.12	0.083	0.062	0.12	Adequate.
F3	Fullerton Ave., East of River	550	60	0.60	0.50	0.17	0.33	0.11	Doubtful.
F4	Fullerton Ave., East of Ashland Ave.	500	70	0.60	0.54	0.17	0.40	0.064	Frequently flooded.
F11	Fullerton Ave., East of Sheffield Ave.	200	70	0.70	0.54	0.213	0.48	0.085	" "
F5	56th St., West of Lake Michigan	*1 100	20	0.30	0.20	0.09	0.08	0.036	" "
F6	51st St., West of Lake Michigan	900	20	0.30	0.20	0.094	0.080	0.066	Barely adequate.
F7	45th St., West of Lake Michigan	500	40	0.40	0.34	0.11	0.20	0.058	Frequently flooded.
F8	41st St., West of Lake Michigan	410	50	0.40	0.40	0.11	0.26	0.07	" "
F9	41st St., West of Prairie Ave.	98	40	0.40	0.34	0.16	0.34	0.17	
F10	Fullerton Ave., West of River	1 010	30	0.50	0.30	0.15	0.17	0.10	
G1	Granville Ave., West of Lake Michigan	425	10	0.20	0.12	0.076	0.05	0.14	Adequate.
G2	Graceland Ave., West of Lake Michigan	160	10	0.10	0.12	0.09	0.07	0.038	Few complaints.
G3	Garfield Boulevard, West of Michigan Ave.	90	20	0.30	0.20	0.16	0.18	0.21	
G4	Garfield Boulevard, East of Michigan Ave.	155	20	0.30	0.20	0.14	0.13	0.14	Complaints due to outlet.
H2	Harrison St., West of River	260	80	0.90	0.60	0.20	0.51	0.20	Adequate.
I1	Indiana Ave., South of Garfield Boulevard	45	20	0.30	0.20	0.20	0.20	0.20	
I3	Irving Park Boulevard, South of Robey St.	208	30	0.30	0.26	0.13	0.19	0.18	Adequate.
L1	Lawrence Ave., West of Lake Michigan	370	10	0.30	0.12	0.11	0.07	0.122	"
L2	Lake Ave., North of 44th Place	26	50	0.40	0.40	0.35	0.47	0.25	Back-water from F7.

* Includes 240 acres of parks.

TABLE 9.—(*Continued.*)

	Location of Sewers.	Area, in acres.	Population per acre.	Impervious area.	C (Estimated.)	Run-off, in Cubic Feet per Second per Acre.		Sewer capacity, in cubic feet per second per acre.	Observation.
						Hill.	Buerger.		
L3	Lawrence Ave., East of Robey St.	162	20	0.30	0.20	0.14	0.14	0.16	Adequate.
L4	Lawrence Ave., West of River	1 800	3	0.30	0.10	0.09	0.03	0.065	Undeveloped.
L5	Lake View Ave., North of Roslyn Place	7.5	50	0.40	0.40	0.48	0.86	0.36	Too small.
M1	Michigan Ave., River to Van Buren	58	500	0.90	0.90	0.29	1.18	0.20	No complaints.
M2	Montrose Ave., West of River	1 450	5	0.10	0.10	0.06	0.03	0.08	Undeveloped.
M3	Michigan Ave., South of Garfield Boulevard	54	20	0.30	0.20	0.19	0.20	0.13	
M4	Montrose Ave., East of Robey St.	170	20	0.30	0.20	0.14	0.15	0.10	Adequate.
M5	Madison St., West of River	280	90	0.90	0.64	0.20	1.03	0.185	"
N1	North Ave., West of River	340	80	0.90	0.60	0.19	0.54	0.25	"
N2	North Ave., East of River	380	80	0.90	0.60	0.18	0.54	0.105	Unknown.
P1	Polk St., West of River	131	120	0.90	0.75	0.24	0.91	0.21	Adequate.
R1	Roscoe St., West of Lake Michigan	170	30	0.40	0.26	0.14	0.21	0.10	Deep basements flooded.
R2	Robey St., North of Montrose Ave.	450	20	0.20	0.20	0.11	0.11	0.077	Undeveloped.
R3	Robey St., North of Byron St.	870	20	0.20	0.20	0.095	0.10	0.068	"
R4	Robey St., North of Roscoe St.	1 260	20	0.20	0.20	0.09	0.08	0.076	"
R5	Robey St., North of Addison St.	1 100	20	0.20	0.20	0.09	0.08	0.054	"
R6	Rush St., North of River	290	90	0.80	0.80	0.19	0.54	0.26	Adequate
R7	Randolph St., West of River	275	60	0.90	0.50	0.20	0.42	0.19	"
S1	Sheridan Road, North of Lawrence Ave.	2 365	10	0.30	0.12	0.078	0.03	0.09	
S2	Sheridan Road, South of Lawrence Ave.	975	25	0.30	0.23	0.09	0.13	0.10	
S3	Surf St., West of Lake View Ave.	50	30	0.50	0.26	0.30	0.30	0.16	
S4	70th St., West of Yates Ave.	1 400	5	0.10	0.10	0.06	0.03	0.093	
S5	73d St., West of Railroad Ave.	1 200	5	0.10	0.10	0.06	0.04	0.08	
T1	West 22d St., West of Western Ave.	3 400	60	0.80	0.50	0.10	0.19	0.074	Occasionally flooded.
T2	35th St., West of Lake Michigan	720	60	0.50	0.50	0.15	0.32	0.104	Frequently flooded.
T3	Thorndale Ave., West of Lake Michigan	185	10	0.20	0.12	0.09	0.07	0.135	Adequate.
T4	West Taylor St., West of River	182	150	0.90	0.80	0.22	0.77	0.29	"
T5	West 12th St., West of River	148	120	0.90	0.70	0.23	0.74	0.35	"
V1	Vincennes Ave., North of 51st St.	50	30	0.30	0.26	0.19	0.28	0.16	"
V2	West Van Buren St., West of River	300	70	0.90	0.54	0.19	0.44	0.18	"
W1	Wentworth Ave., North of Calumet River	5 500	5	0.20	0.10	0.05	0.02	0.06	Undeveloped.
W2	Western Ave., North of North Branch	700	5	0.10	0.10	0.07	0.04	0.06	"
W3	Wabash Ave., South of Garfield Boulevard	54	20	0.30	0.19	0.19	0.19	0.31	
W4	Webster Ave., East of River	200	50	0.50	0.40	0.21	0.36	0.20	Many complaints.
W5	Webster Ave., East of Halsted St.	65	60	0.70	0.48	0.28	0.56	0.24	" "

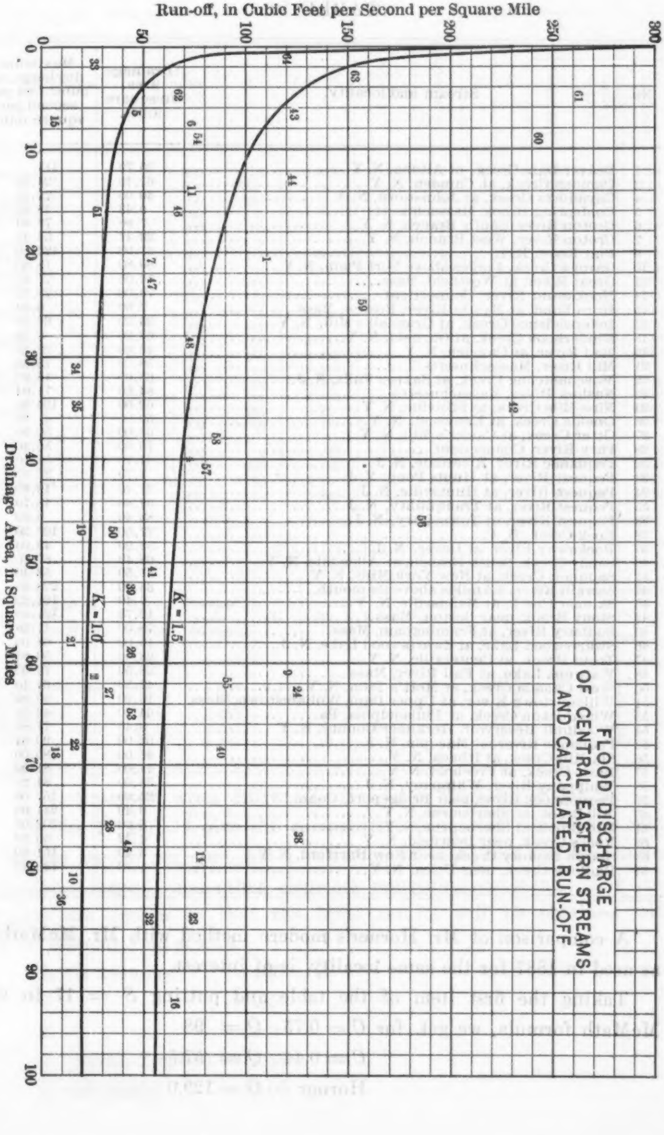

FLOOD DISCHARGE
OF CENTRAL EASTERN STREAMS
AND CALCULATED RUN-OFF
Run-off, in Cubic Feet per Second per Square Mile
Drainage Area, in Square Miles
K = 1.5
K = 1.0

FIG. 1.

TABLE 10.

No.	Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.
1	Beaver Dam Creek, at Altona, N. Y.	20.70	111
2	Camden Creek, at Camden, N. Y.	61.40	24.10
4	Cayadutta Creek, at Johnstown, N. Y.	40.00	72.40
5	Coldspring Brook, Massachusetts	6.43	48.40
6	Croton River, South Branch, N. Y.	7.80	73.90
7	Croton River, West Branch, N. Y.	20.47	54.40
9	Flat River, R. I.	61.00	120.90
10	Garoga Creek, 4 miles above Fort Plain, N. Y.	80.80	15.80
11	Great River, at Westfield, Mass.	14.00	71.40
14	Hockanum River, Connecticut	79.00	78.10
15	Horn Pond, at Mystic River, Boston, Mass.	7.60	6.50
16	Independence Creek, at Crandall's Mill, N. Y.	93.20	66.50
18	Kinderhook Creek, at Garfield, N. Y.	68.20	9.00
19	Mad River, at Camden, N. Y.	46.60	22.10
21	Mill River, Massachusetts	58.00	15.50
22	Musconetcong Creek, at Saxton Falls, N. J.	68.00	15.90
23	Nashua River, Massachusetts	84.50	71.04
24	Nine-Mile Creek, at Stillville, N. Y.	62.60	124.90
26	Oneida Creek, at Kenwood, N. Y.	59.00	41.20
27	Otter Creek, at Caster's Mill, N. Y.	63.00	30.90
28	Park River, Connecticut	76.00	30.40
32	Pequanac River Riverdale, N. J.	84.70	52.50
33	Pequest River, at Hunts Pond, N. J.	1.70	25.30
34	Pequest River, at Huntsville, N. J.	31.40	19.80
35	Pequest River, at Tranquillity, N. J.	34.80	18.70
36	Pequest River, at Townsbury, N. J.	83.40	9.60
38	Rock Creek, D. C.	77.50	126.30
39	Rockaway River, at Dover, N. J.	52.20	43.00
40	Sandy Creek, South Branch, at Allendale, N. Y.	68.40	87.70
41	Sauquoit Creek, at New York Mills, N. Y.	51.50	53.40
42	Sawkill River, 4.5 miles above its mouth	35.00	228.60
43	Skinner Creek, at Mannsville, N. Y.	6.40	124.20
44	Stony Brook, near Boston, Mass.	12.73	121.00
45	Sudbury River, at Framingham, Mass.	78.00	41.38
46	Swartswood Lake, at Swartswood Lake, N. J.	16.00	68.00
47	Trout Brook, at Centerville, N. Y.	23.00	50.60
48	Watuppa Lake, at Fall River, Mass.	28.50	72.00
50	West Canada Creek, at Mott's Dam, N. Y.	47.50	34.10
51	Williamstown River, at Upper Dam, Williamstown, Mass.	16.20	30.90
53	Wissahickon Creek, at Philadelphia, Pa.	64.60	43.50
54	Woodbull Reservoir, Herkimer County, N. Y.	9.40	77.80
55	Pequanac River, at Macopin, N. J.	62.00	90.84
56	Six-Mile Creek, at Ithaca, N. Y.	46.00	185.00
57	Basin Creek, at Freehold, N. Y.	41.00	81.20
58	Whippany River, Whippany, N. J.	38.00	84.20
59	Pequonnock River, near Bridgeport, Conn.	25.00	157.00
60	Mill Brook, at Sherbourne, N. Y.	9.40	241.00
61	Mill Brook, at Sherbourne, N. Y.	5.00	262.00
62	Reel's Creek, near Deerfield, N. Y.	4.42	66.90
63	Starch Factory Creek, near New Hartford, N. Y.	3.40	151.60
64	Budlong Creek, near Utica, N. Y.	1.18	120.40

A comparison of Mr. Horner's modern method with Mr. McMath's, as used in 1887 for the same locality, is of interest.

Taking the first item of the table and putting $S = 17$ in the McMath formula, we get, for $C = 0.75$ $Q = 98$

" $C = 0.44$ $Q = 57.5$

Horner $Q = 129.0$

TABLE 11.—WALWORTH SEWER, CLEVELAND, OHIO.

Sub-main.	L, length, in feet.	A.	f or C.	s, also called S.	m, assumed.	R.	K.	Q. Parmley.	Q. Bürkli-Ziegler.	Q. Buerger.
Gordon, N	1 600	12	0.625	5	9	4	1.1	30	24	18
" S	5 800	123	"	10	16	....	....	245	164	172
Clark, W	6 600	350	"	7	9	....	....	540	337	455
Alum, N	1 300	8	"	6	9	....	....	22	19	13
" S	4 800	53	"	10	16	....	....	121	87	74
Purdy	1 300	9	"	6	9	....	....	25	20	14
Waverly	1 300	9	"	6	9	....	....	25	20	14
Guernsey	700	8	"	6	9	....	....	22	19	14
Swiss, N	800	75	"	6	9	....	....	144	100	135
" S	600	6	"	8	16	....	....	19	16	11
Junction, N	2 900	71	"	4	9	....	....	124	86	100
" S	7 200	187	"	7	9	....	....	320	206	235
Burton, N	2 600	57	0.75	4	9	....	....	123	88	97
" S	9 800	466	0.625	5	9	....	....	630	375	540
Pollock	1 200	19	0.625	5	9	....	....	44	34	31
Mill, N	700	42	0.75	4	9	....	....	96	58	106
" S	13 900	797	0.625	3	9	....	....	858	494	718
Pearl, N	4 100	154	0.75	4.	9	....	....	282	185	245
" S	2 400	58	0.75	3	9	....	....	117	83	98
Brevier, N	600	......	......	......	......	....	....	......	......	......
" S	600	4	0.625	15	25	....	....	14	14	8
Kenilworth	11 150	416	0.625	3	9	....	....	500	303	405
Scranton	2 000	26	0.75	40	49	....	....	115	87	61
Cliff	3 400	32	0.625	3	9	....	....	59	44	42

TABLE 12.

Location of sewer, M. H. to M. H.	Area, in acres.	Length, L, in feet.	C.	S.	m.	K.	Q, in Cubic Feet per Second. Horner.	Q, in Cubic Feet per Second. Buerger.
153 to main line	61.4	3 800	0.44	17	20	1.7	129.0	117.0
175 to 165	3.15	950	"	7	10	"	6.9	5.7
162 to 161	14.9	1 600	"	8	10	"	32.7	25.3
157 to b	33.5	2 600	"	15	15	"	73.8	63.7
c to 155	48.9	3 200	"	19	20	"	105.0	92.9

Comparison with Bürkli-Ziegler and McMath Results.

Tables 13 and 14 give the comparative computed run-off, in cubic feet per second per acre, for a variation of area, and for a variation in slope, respectively, other elements remaining constant. The constants are chosen so that the same results will be obtained with $A = 100$ and $S = m = 4$.

L (is assumed) $= 400 \sqrt{A}$

C $= 0.50$

K $= 1.0$

R (Bürkli-Ziegler) $= 3.58$

R (McMath) $= 3.03$

TABLE 13.

$S = m = 4.$

Area, in acres.	q = Run-Off, in Cubic Feet per Second per Acre.		
	Bürkli-Ziegler.	McMath.	Buerger.
10	1.42	1.27	1.00
50	0.95	0.92	0.85
100	0.80	0.80	0.80
200	0.67	0.70	0.78
500	0.54	0.58	0.68
800	0.48	0.53	0.62
1 000	0.45	0.50	0.59
2 000	0.38	0.44	0.54
5 000	0.30	0.36	0.46
8 000	0.27	0.33	0.41
10 000	0.25	0.32	0.38

(Slide-rule work.)

TABLE 14.

$A = 100.$

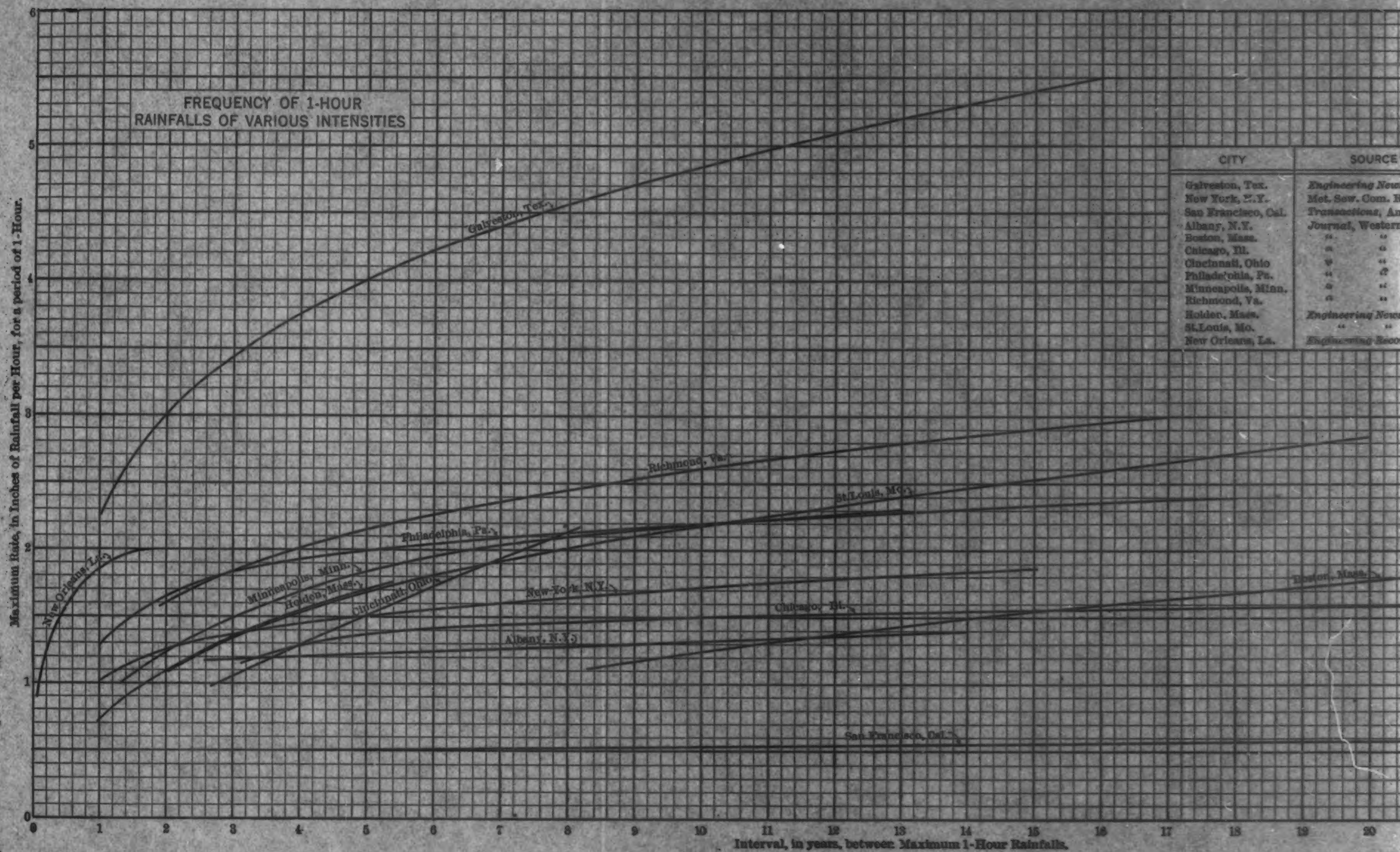
$S = m$	q = Run-Off, in Cubic Feet per Second per Acre.		
	Bürkli-Ziegler.	McMath.	Buerger.
3	0.75	0.75	0.76
4	0.80	0.80	0.80
5	0.85	0.83	0.85
7	0.92	0.88	0.95
10	1.00	0.96	1.00
15	1.11	1.04	1.05
20	1.20	1.10	1.10
30	1.32	1.19	1.20
40	1.42	1.26	1.25
50	1.50	1.32	1.30
60	1.57	1.37	1.35
80	1.69	1.46	1.38

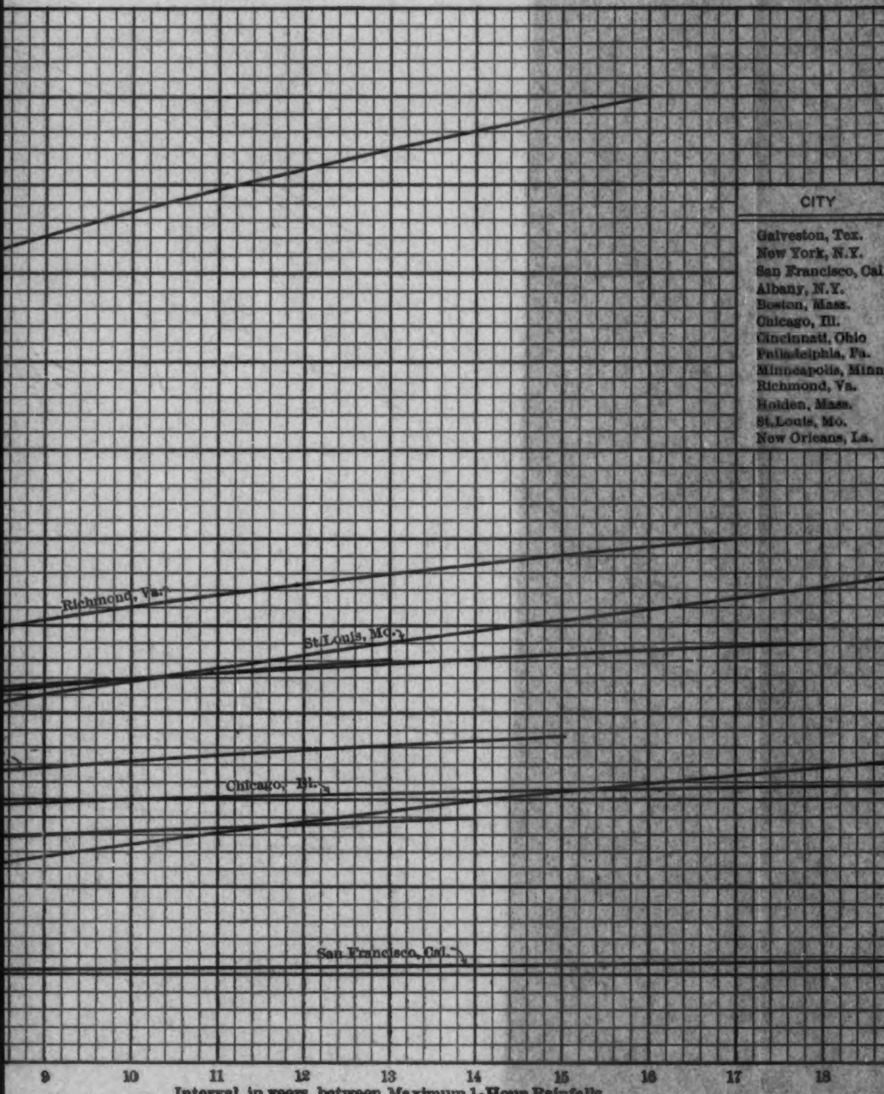
(Slide-rule work.)

Tables 13 and 14 are of use only to those who think in terms of either of these two formulas.

Determination of C.—The value of C, which, in the writer's formula, represents the ratio between the run-off and the total rainfall, or, in other words, the ratio of that part of the rainfall not absorbed by or retained on the soil to the total rainfall, will vary much with special local conditions. A study of the nature of the land and its improvements will give some reasonable conclusions as to C.

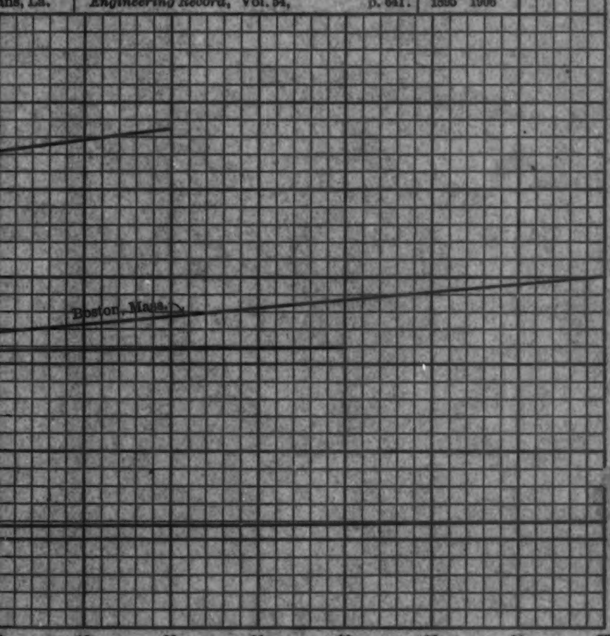
The existing improvements are not always to be used as a basis of storm-water drain design in a growing community, and for such a


FREQUENCY OF 1-HOUR
RAINFALLS OF VARIOUS INTENSITIES
Maximum Rate, in Inches of Rainfall per Hour, for a period of 1-Hour.
Interval, in years, between Maximum 1-Hour Rainfalls.
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
Galveston, Tex.
Richmond, Va.
St.Louis, Mo.
Philadelphia, Pa.
Minneapolis, Minn.
Holden, Mass.
Cincinnati, Ohio
New Orleans, La.
New York, N.Y.
Chicago, Ill.
Albany, N.Y.
Boston, Mass.
San Francisco, Cal.
CITY
SOURCE
Galveston, Tex.
New York, N.Y.
San Francisco, Cal.
Albany, N.Y.
Boston, Mass.
Chicago, Ill.
Cincinnati, Ohio
Philadelphia, Pa.
Minneapolis, Minn.
Richmond, Va.
Holden, Mass.
St.Louis, Mo.
New Orleans, La.
Met. Sew. Com.
Transactions,
Journal, Western
" "
" "
" "
" "
" "
" "
Engineering
" "



CITY
Galveston, Tex.
New York, N.Y.
San Francisco, Cal
Albany, N.Y.
Boston, Mass.
Chicago, Ill.
Cincinnati, Ohio
Philadelphia, Pa.
Minneapolis, Minn
Richmond, Va.
Holden, Mass.
St.Louis, Mo.
New Orleans, La.
Richmond, Va.
St.Louis, Mo.
Chicago, Ill.
San Francisco, Cal.
9
10
11
12
13
14
15
16
17
18
Interval, in years, between Maximum 1-Hour Rainfalls.

TY	SOURCE OF INFORMATION		PERIOD
, Tex.	*Engineering News*, Vol. 71, Jan. 8, 1914,	p.69.	1898 1913
, N.Y.	Met. Sew. Com. Report 9, Nov. 1913,	p. 8.	1896 1911
cisco, Cal.	*Transactions*, Am. Soc. C.E., Vol. 65,	p. 312.	1908
N.Y.	*Journal*, Western Soc. Engrs., Vol. 18,	p. 672.	1899 1912
Iass.	" " " " " "	p. 674.	1879 1904
Ill.	" " " " " "	p. 675.	1889 1910
i, Ohio	" " " " " "	p. 676.	1897 1912
hia, Pa.	" " " " " "	p. 679.	1895 1912
lis, Minn.	" " " " " "	p. 677.	1899 1912
l, Va.	" " " " " "	p. 680.	17 years
Iass.	*Engineering News*, Vol. 69,	p. 704.	1897 1912
Mo.	" " Vol. 64,	p. 328.	1890 1909
ans, La.	*Engineering Record*, Vol. 54,	p. 641.	1895 1906



section, not fully developed, some estimate must be made of the probable ratio, C, corresponding to an assumed development at some future time.

A fact, not always fully recognized, is that rain-water falling on a fully impervious surface does not of necessity reach the sewers. This is particularly likely to be the case in a moderately populated district. Rain-water falling on roofs will fully run off from these roofs, but it will often be discharged on lawns and other unprotected surfaces, and the net run-off from these roofs will accordingly be little if any greater than if the rain fell on an undeveloped area of the same imperviousness as the surrounding soil. The same is true of rain-water from paved walks, which are usually bordered by grassed earth of large extent compared with the walks themselves, and the discharge on these unprotected surfaces is largely absorbed. For heavily populated districts, arrangements usually permit of direct discharge of roof and pavement water to the sewers, and no such reduction of run-off need be made.

A very weighty factor in the value of C is the perviousness of the soil, and the corresponding run-off from unprotected areas. The theoretical range will run from 0 for light sandy soils to 1.00 for some rocky or clayey soils. For less unusual conditions, and for the ordinary range of practice, the writer assumes a range of from 5% for soils of sandy nature up to 50% for clay soil. Based on these figures and assumptions, as to the type, number, and size of buildings, areas of streets, walks and their character, the curves of Fig. 2 are derived, in which C is expressed as a function of the population per acre. Table 15 gives a comparison of estimated values of C, based on population, from a number of sources. The coefficient, C, probably does not, in all cases, represent the identical conception.

Determination of K.—The determination of K, the rainfall lasting for a period of 60 min., is a relatively simple matter. Records for shorter periods are not always available, but 60-min. rainfalls are often to be had. Plate XVI gives the rate of rainfall for 1 hour duration, in a number of cities, plotted as a function of the average interval in years between such rainfalls. Having a curve of this character for any locality, a selection of the frequency with which the drains are to run full will determine the value of K to be used.

TABLE 15.

Population per acre.	Kuichling.*	Butt.†	Grunsky.‡	Imhoff.§	Buerger (average).
10	0.10	0.16	......	0.10	0.21
20	0.20	0.26	0.30	0.15	0.27
30	0.28	0.34	0.40	0.20	0.33
40	0.38	0.44	0.50	0.25	0.39
50	0.47	0.55	0.55	0.31	0.45
60	......	0.64	0.60	0.37	0.51
70	......	0.74	0.65	0.42	0.57
80	......	0.84	0.70	0.48	0.61
90	......	0.94	0.73	0.54	0.66
100	......	1.00	0.75	0.60	0.69

* H. N. Ogden, M. Am. Soc. C. E., "Sewer Design," 1909, p. 62.
† E. E. Wallington Butt, *The Surveyor*, 1907, Vol. 32, p. 133.
‡ C. E. Grunsky, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXV, p. 319.
§ K. Imhoff, "Taschenbuch für Kanalisations-Ingenieure," 1912, p. 17.

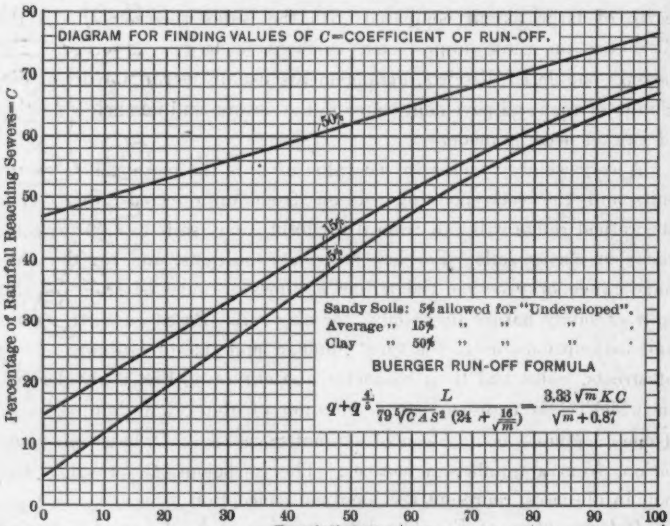
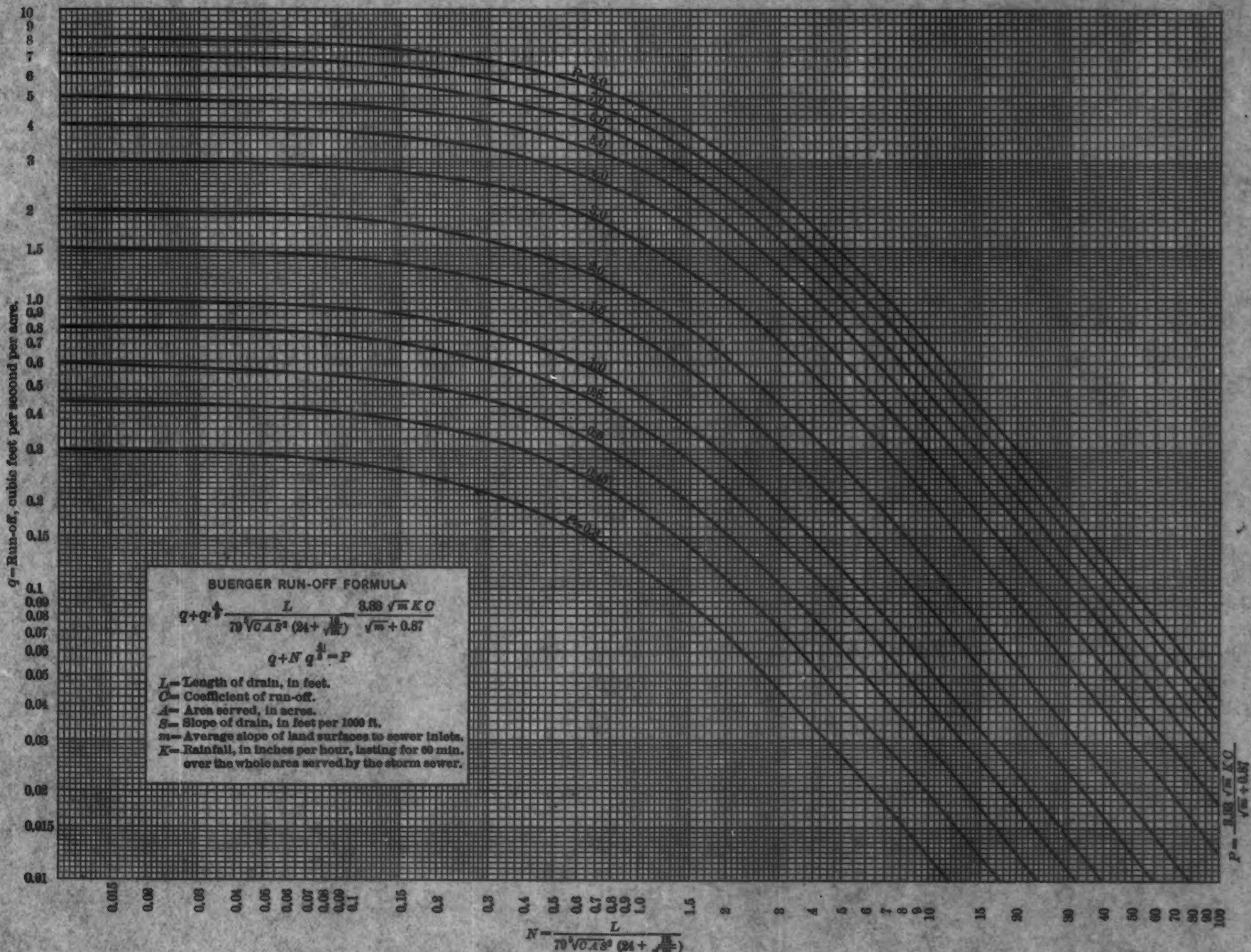


FIG. 2.

Mr. Imhoff suggests* that an assumption that the sewers will be fully charged by the run-off once every year is a reasonably economical and practical basis, and that longer intervals are not warranted. This would correspond to a value of K ranging from 1 to 1.25 for the Central Eastern States, as shown by Plate XVI.

* *Loc. cit.*, p. 19.

PLATE XVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1325.
BUERGER ON
STORM-WATER RUN-OFF.



The rainfall for which the drains are to provide and the intervals between full flow, must be matters of judgment to be decided according to the relative importance of adequate drainage and first cost.

There is no exact basis on which such a comparison can be made. The amount of damage resulting from a flooding of storm sewers cannot be gauged accurately in advance in dollars and cents. In a few cases the loss resulting from any flooding is so obviously heavy that it immediately appears necessary to provide drains of ample capacity to prevent such flooding under the worst possible conditions. For more ordinary cases, the resultant damage is moderate, and, if not so frequent that the annoyance of flooding must enter as a serious factor, it may be better to submit to such occasional losses than to endure the heavier first cost of drains of larger size. Each case must rest on its own merits, and it is not unreasonable in one city to have an allowed interval between full flow in the drains of from 6 months to 20 years for districts of different character.

Solution of Proposed Formula.—After the selection of values for C and K, which requires some degree of judgment, the work of determining the run-off for any districts of known area, sewer length, sewer grade, surface, and slope becomes entirely mechanical.

To aid in the ready solution of the formula, three diagrams are given. The first of these, Fig. 3, gives the value of P, the right-hand member of the equation. The second diagram, Fig. 4, gives the value of N, the second term of the left-hand member. The third diagram, Plate XVII, gives the values of q, as a function of N and P.

TYPICAL APPLICATION OF RUN-OFF FORMULA.

This calculation is made for one district in Altoona, Pa. A storm curve of maximum intensity is assumed (not here reproduced).

$$C = 0.50$$
$$A = 130 \text{ acres.}$$

Rational Method.—Time of concentration:

1 800 ft. of 24-in. sewer on 80-ft. slope,	$1\frac{1}{2}$ min.
1 800 " " 54-in. " " 18-ft. "	2 min.
Time to reach sewers..................	5 min.
	$8\frac{1}{2}$ min.

Intensity for $8\frac{1}{2}$ min. duration = 6.2 in. per hour.

$Q = 130 \times 6.2 \times 0.5 = 403$ cu. ft. per sec.

Buerger Formula.—

$L = 3\,600,$

$A = 130,$

$C = 0.50,$

$K = 2.3$ (from rainfall curve),

$m = 49$ (from map),

$S = 40$ (" "),

P, from Fig. $3 = 3.4$,

N, from Fig. $4 = 0.158$,

q, from Plate XVII $= 3.0$,

$Q = 130 \times 3.1 = 390$ cu. ft. per sec.

DIAGRAM FOR FINDING VALUES OF THE EXPRESSION,

$$P = \frac{3.33\sqrt{m}\,KC}{\sqrt{m}+0.87}$$

IN THE BUERGER RUN-OFF FORMULA

$$q + q^{\frac{4}{5}} \frac{L}{79\sqrt[5]{CAS^2}\left(24+\frac{16}{\sqrt{m}}\right)} = \frac{3.33\sqrt{m}\,KC}{\sqrt{m}+0.87}$$

C=Coefficient of run-off.
m=Average slope of land surfaces to sewer inlets.
K=Rainfall, in inches per hour, lasting for 60 min.

Find value of m at bottom of diagram. Run up this line as indicated by dotted line to curve. At this point run horizontally to K line then vertically to C line, then run horizontally to P values at right hand side.

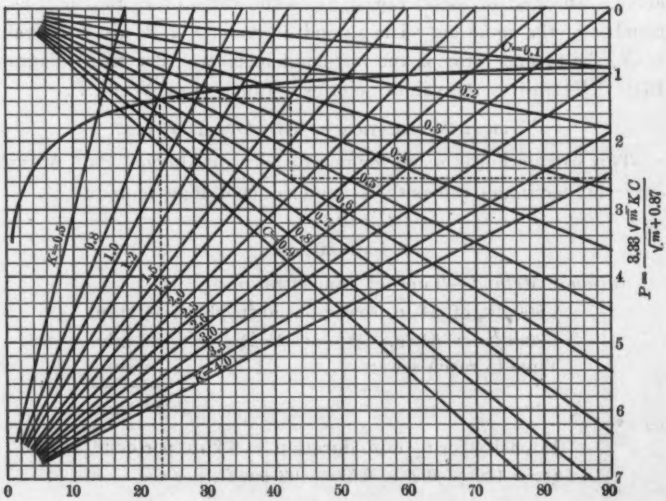


FIG. 3.

DIAGRAM FOR FINDING VALUES OF THE EXPRESSION,

$$N = \frac{L}{79\ \sqrt[5]{CAS^2}\,(24 + \frac{16}{\sqrt{m}})}$$

IN THE BUERGER RUN-OFF FORMULA:

$$q = q^{\frac{4}{5}}\frac{L}{79\sqrt[5]{CAS^2}\ (24 + \frac{16}{\sqrt{m}})} = \frac{3.33\sqrt{m}\ \ KC}{\sqrt{m} + 0.87}$$

L = Length of drain, in feet.
C = Coefficient of run-off.
A = Area served, in acres.
S = Slope of drain, in feet per 1000 ft.
m = Average slope of land surfaces to sewer inlets.

Find value of m at bottom of diagram, run vertically as indicated by dotted line to C curve. At this point run horizontally to S line then vertically to A line then horizontally to L curve, then run up vertically to N values at top of diagram.

$$N = \frac{L}{79\ \sqrt[5]{CAS^2}\,(24 + \frac{16}{\sqrt{m}})}$$

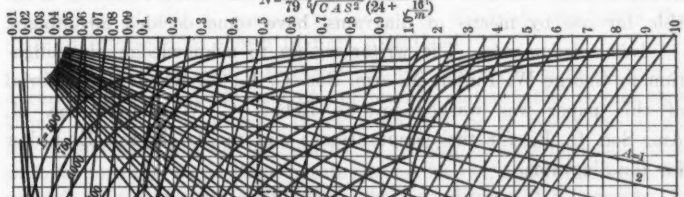


FIG. 4.

The writer has had a large number of computations made on areas varying in imperviousness, extent, length, slope, and with various rates of rainfall, and can state, without qualification or limitation, that the proposed formula gives results in all cases reasonably close to those which will be obtained by the rational method, and that the formula can be taken as an acceptable equivalent.

As compared with the exponential formulas, the results show no relationship except an occasional coincidence at some points under suitable combinations of conditions.

In the writer's judgment, the correctness and usefulness of any run-off formula must stand on its corroboration of results obtained by the rational method. The new formula fully meets this condition. Because of its complicated form, there will no doubt be many who will consider a graphical or cut-and-try method more useful. The direct results, which can be obtained from the formula readily available for use by means of diagrams, have some decided advantages.

Acknowledgments.—The writer wishes to acknowledge his obligation to George W. Fuller, M. Am. Soc. C. E., for whom the material of this paper was worked up; to Robert C. Wheeler, Assoc. M. Am. Soc. C. E., for the computations; and to Mr. Myron E. Fuller for the diagrams.

DISCUSSION

R. C. STRACHAN,* M. AM. SOC. C. E.—The work of the author in gathering and sifting data for comparing the results obtained from numerous run-off formulas, and in epitomizing the influences of the many factors in a new formula taking into account soil conditions, grades, rainfall, density of population, and extent of territory drained, is one of great magnitude, and is of permanent value to the designing engineer, even though, in his judgment, the new formula may not be universally applicable. Mr. Strachan.

Such a formula is necessarily far from simple, and its practical utility must depend largely on the diagram or other device by which ease and celerity of solution are promoted.

It cannot be said that simplicity is the dominant feature of the diagrams accompanying the paper; and it is unfortunate that Mr. Buerger did not make use of the nomographic method of charting, by which Plate XVII could be reduced to three lines and the diagrams for finding P and N constructed so as to eliminate absolutely the maze of intersections shown in Figs. 3 and 4.

KENNETH ALLEN,† M. AM. SOC. C. E. (by letter).—The effort to find a run-off formula, both easy of application and reliable, is one that should meet with approval. If we could only be in possession of all the factors affecting the phenomena of run-off, its determination by a rational formula would leave nothing to be desired, and it has been the writer's opinion for some time that where conditions of topography or climate vary to a material degree from those on which such empirical formulas as the Bürkli-Ziegler are based, it would not be safe to use any but the rational method. The great difficulty here lies in the fact that the data necessary for its proper application—such as maximum intensity of precipitation and time of concentration—are so rarely known; and, if they are to be guessed at, why not use some more convenient empirical formula? Mr. Allen.

The weak point in formulas of the Bürkli-Ziegler type lies in their failure to provide for differences in the time of concentration due to the shape of the drainage area. They all have the merit of easy application, but, as the author points out, they were (excepting the Gregory formula) developed from local data, and, therefore, were not necessarily applicable to other places.

In any case, lack of care in the selection of proper values for C and R may result in large errors. In particular, the value of R appears to be subject to different interpretations by different engineers. It is usually stated—as by the author—as "rainfall, in inches per hour", but

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* Richmond Hill, N. Y.

† New York City.

Mr. Allen. it is frequently taken as the rate of precipitation, in inches per hour for 10, 20, or 30 min., or for the probable time of concentration for the area in question. If we determine the rate of precipitation by Formula (13) this, and consequently Formula (9), will be twice as great if $t = 20$ min. as if $t = 60$ min. The importance of definite understanding on this point is evident. The value to be assigned to R or I depends on the long-continued records of a self-registering gauge on or near the drainage district.

The determination of R for the conditions of maximum run-off, or I, involves the time for concentration, which in turn depends on the hydraulics of the drainage vehicle, the shape of the area drained, and the distribution of the rain on this area. These present a difficult problem to be solved for each sewer district, if accuracy is attempted, and the results of rain-gauge observations and sewer gaugings for long periods are necessary for its solution. Those who have attempted it know the care necessary to determine the correct discharge of a sewer in times of storm, where surface levels vary without apparent cause and friction coefficients can only be known by special experiments. One factor, which applies to large areas, that is often overlooked, is the decrease in average maximum intensity as the area increases. This factor has been called a "distribution coefficient" by Messrs. Wynne-Roberts and Brockmann,* who adopt a value proposed by Frühling expressed by the formula, $C = 1 - 0.002 \sqrt{L}$, in which L is the length, in feet, of the longest line of sewer above.

The author reduces all values of R to that for 60 min., which he calls K, and then deduces his empirical formula to conform as well as may be with the results of actual gaugings. Besides the merit of being definite, in this respect, there is the further advantage in the use of K, pointed out by the author, that there are many more records available for 1 hour than for shorter periods.

Whether the relation of the maximum hourly rate to that for the shorter periods on which the capacity of sewers is usually based is constant for different locations seems to be improbable, and, for this reason, a formula using the shorter period, or one based on the time for concentration, would appear preferable.

One difficulty has been met in determining high rates of precipitation for short intervals with most of the recording gauges in use due to the necessarily small scale of hours ($\frac{1}{2}$ in. or less per hour). This has been ingeniously overcome by George A. Carpenter, M. Am. Soc. C. E., by causing a jar to be given to an arm carrying the recording pen every 5 min. by the clock mechanism. This produces a short horizontal line across the line on the chart which, at such times, approximates the vertical, and the precipitation for any 5-min. interval can then be scaled off quite correctly between any two consecutive horizontal lines.

* *Canadian Engineer*, Vol. 24.

Mr. Allen.

The selection of K, or the frequency with which surcharge of sewers may be allowed, is an important and delicate matter, involving undue cost of construction on the one hand and the possibility of loss and consequent damage suits from flooding basements on the other. The assumption of a value of K corresponding to 12 months suggested by Dr. Imhoff as reasonable is, in the writer's opinion, far too short a period for well-developed conditions in large cities, where 10 years or more would often be warranted.

The writer has not examined the results obtained by the proposed formula sufficiently to express an opinion on its reliability under different conditions. It is admittedly somewhat awkward of direct application, but this practical difficulty is overcome by the use of diagrams, which is a sensible procedure in almost any case. Referring to Plate XVI, giving the frequency of 1-hour rainfalls of different intensities, it may be well to mention that the New York curve is based on records of the greatest single intensity for each month of observation, and that other high intensities were not recorded. For this reason, it is probable that the frequencies shown by this curve are slightly below the actual.

The author is to be congratulated on having produced, after so careful a study, a formula free from some of the objections found in those proposed heretofore.

Mr. Horner.

W. W. Horner,* Assoc. M. Am. Soc. C. E. (by letter).—The author presents a formula for storm-water run-off and introduces it as "empirical". He speaks of the "so-called rational method" and of the "formula method", and states "that the proposed formula gives results in all cases reasonably close to those which will be obtained by the rational method, and that the formula can be taken as an acceptable equivalent". Elsewhere, he speaks of the rational method as involving "cut-and-try methods" and as being "relatively laborious".

The author does not include in his paper any information in regard to the development or construction of his formula. The writer became interested in the manner of appearance of the variables in the new formula, and found that, for an ideal condition of area and slope, and with the type of rainfall curve which the author uses in comparing gaugings, he was able to derive from the theory of the "rational method" a formula very similar to that of the author. For this reason, the writer is of the opinion that the new formula should be described as a rational formula for ideal conditions.

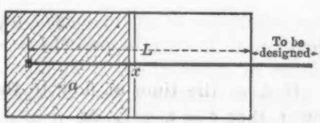

Fig. 5.

* St. Louis, Mo.

Mr. Horner. The derivation mentioned is of considerable interest, and is presented as follows: Assume conditions as shown in Fig. 5. A circular sewer having a grade of S feet per thousand, a length, L, and a drainage area at the lower end of A. Let a equal the area drained at any point, x, the sewer to be uniformly proportioned to carry the run-off at all points, and the tributary area to be proportional to the length; then the run-off at x will be

$$Q_x = a\,I\,c = 4\,r^2\,B\sqrt{r\,\frac{S}{1\,000}},$$

where r is the hydraulic radius at that point, and B is the coefficient of the Chezy formula.

Eliminating r from these two gives

$$V_x = \frac{B^{\frac{4}{5}}\,S^{\frac{2}{5}}}{26.4}\,(a\,I\,c)^{\frac{1}{5}}.$$

The time of flow may be expressed from

$$d\,t = \frac{d\,(L)}{V_x},$$

or, where

$$\frac{d\,(L)}{d\,(a)} = \frac{L}{A},$$

$$d\,t = \frac{L}{A\,V_x}\,d\,(a).$$

Substituting V_x from above, integrating between $a = 0$ and $a = A$, and dividing by 60 to reduce to minutes:

$$t_L = \frac{0.55\,L}{(C\,I)^{\frac{1}{5}}\,B^{\frac{4}{5}}\,\sqrt[5]{S^2\,A}} = \frac{0.55\,L}{q^{\frac{1}{5}}\,B^{\frac{4}{5}}\,\sqrt[5]{S^2\,A}}$$

assuming a rainfall curve of the form

$$I = \frac{90\,K}{t + 24},$$

then

$$q = C\,I = \frac{80\,K\,C}{t + 24}.$$

If t_i = the time of flow to the inlet and t_L the time through the sewer, then $t = t_i + t_L$, or

$$q = \frac{80\,K\,C}{t_i + t_L + 24}.$$

Substituting the value of t_L above and solving for q,

$$q + q^{\frac{4}{5}}\,\frac{L}{1.82\,(24 + t_i)\,B^{\frac{4}{5}}\,\sqrt[5]{S^2\,A}} = \frac{80\,K\,C}{24 + t_i}$$

The velocity of flow to the inlet may be assumed to be proportional to the square root of the surface slope, or $V_i = x\,\sqrt{m}$.

Mr. Horner.

If l = an average distance from the water-shed to the inlet, then

$$t_i = \frac{l}{x\sqrt{m}}.$$

For average conditions, assume $l = 500$, $m = 10$, and $t = 5$.

Then $$\frac{l}{x} = 15.5, \text{ say } 16,$$

and $$t_i = \frac{16}{\sqrt{m}}.$$

Also, assume an average value of the Chezy coefficient, B, as 110; then the formula becomes

$$q + q^{\frac{4}{5}} \frac{L}{78\sqrt[5]{S^2 A}\left(24 + \frac{16}{\sqrt{m}}\right.} = \frac{3.33\sqrt{m}\ K\ C}{\sqrt{m} + 0.67}.$$

This formula, though based on several averages and assumptions, is a rational one; it differs slightly from the author's formula in the numerical coefficient and in the appearance of C under the radical with $S^2 A$. This appearance of C is irrational, and may warrant the use of the word empirical as describing the formula. The term, C, appearing as a one-fifth power, however, has little effect on the results, because, for a range of C from 0.3 to 0.8, $C^{\frac{1}{5}}$ varies only from 0.79 to 0.96.

If an average value of 0.9 were introduced into this formula, it would become

$$q + q^{\frac{4}{5}} \frac{L}{86\sqrt[5]{C S^2 A}\left(24 + \frac{16}{\sqrt{m}}\right)} = \frac{3.33\sqrt{m}\ K\ C}{\sqrt{m} + 0.67},$$

which is almost identical with that of the author. If other values of B, C, and t_i had been assumed, the numerical coefficients would not have agreed, but the manner of appearance of the variables would have been the same.

The writer wishes to ask why the coefficient, C, is called the ratio of "run-off to total rainfall"? The coefficient of run-off is generally considered as the ratio of the rate of run-off to the intensity of rainfall producing that rate. This latter definition has a definite meaning in sewer work, and the effect of changes in it are readily understood. The C, as defined by the author, is of importance in water-supply work, but does not appear to have any known relation to the rate of run-off.

Whatever the origin of the formula, it may be seen from the foregoing derivation that it may be expected to give reasonable results for certain average conditions. The conditions met in actual practice, however, will vary widely from any mean. In the main sewer of any large system, frequent changes of size and grade, of surface conditions, and even of soil conditions, will be encountered. By the rational

Mr. Horner. method, all these items will enter in their proper place and be taken into account; in any formula, only a general mean condition can be considered, and, in any particular case, the results may vary widely.

The writer is of the opinion that the formula will not give results in all cases reasonably close to those obtained by the rational procedure, and that in general it will not effect any economy of labor in design. In the new formula, all the data and the same analysis of them are required as for the rational method. For the purposes of design, a sewer is divided into a number of sections, according to the areas drained and the grades. In the rational procedure, the grade of a section enters into the calculation of the time of flow for the section, and has no relation to the grades of other sections; in applying the formula to each section, an average of all grades above is required. Again, the variable, m, in the rational method, affects only the initial inlet time, but in the formula it enters into each solution.

The writer has found the procedure of design by the rational method simple, direct, and comprehensible at every step. Although its application to a single unit of sewer in undeveloped territory may call for cut-and-try methods, in the design of a system of sewers, the procedure is definite, the design of each unit leading directly to the next. This is shown clearly in the tabulation for St. Louis conditions referred to by the author.*

The writer does not wish to discount the value of the new formula as a formula, for it is undoubtedly a great advance over the older ones cited in the paper, but wishes to emphasize his belief that the practice in sewer design has already advanced beyond the stage in which the whole problem may be solved satisfactorily by a single mathematical expression of any kind.

Mr. Sherman. CHARLES W. SHERMAN,† M. AM. SOC. C. E.—The speaker agrees fully with Mr. Buerger's statement, "It is almost everywhere conceded that the rational system * * *, if used with proper discretion, will give very satisfactory results", but he is inclined to different views on many of the other statements in the paper. He believes that Mr. Buerger is mistaken as to the methods of determining storm-water run-off which are receiving most favor in the larger and better organized engineering departments, and he feels that, in presenting another empirical formula, unless it shall give results very closely paralleling those of the rational method (the possibility of which the speaker doubts), he is tending to foster a method of design which is irrational.

The author suggests as one of the reasons that, in his opinion, an empirical formula is likely to be more popular in such design than the direct application of the rational method, that the latter is rela-

* *Engineering News*, 1910, Vol. 64, p. 326.

† Boston, Mass.

Mr. Sherman.

tively laborious, and "requires, in addition, a material exercise of judgment". With regard to the comparison of the labor required in the solution of the true rational method and the use of the suggested formula, it is evident that the same preliminary work of determining drainage areas, laying out sewer lines, estimating grades, etc., must be done, in any event; and an examination of the diagrams presented with the paper indicates at least a possibility that finding a value of P by following out four lines in one complex diagram, and a value of N by similarly following five lines in another and more complex diagram, and finally, by determining the location of these two values on a third diagram, to obtain the required run-off figure, would be very nearly as laborious as the direct solution of the rational method, besides being accompanied with a greater likelihood of error caused by misreading the diagrams. Disregarding this point, however, does the author wish us to believe that an engineer, not sufficiently familiar with the territory under consideration and with methods of design to be trusted to exercise his judgment in the use of the rational method, can proceed to design storm-water sewers by the use of the proposed formula and diagrams? The speaker is of the opinion that no one without ample understanding of the subject should be entrusted with matters of such importance, and that putting a formula or diagrams in the hands of inexperienced or untrained men and leaving them to obtain results by substitution of values is not to be commended.

Although the author does not claim definitely that the comparison of his formula with the older empirical formulas, such as that of McMath, has any bearing on the adequacy of a method which is intended to give figures closely paralleling those of the rational method, he submits such comparisons in such a way as to lead to their use as a gauge of his formula. The speaker does not think that this comparison should be taken as significant, and, accordingly, he feels that the presentation of the older empirical formulas is out of place in this paper. He would call attention to the fact, however, that the Hering formula, representing the New York diagrams, as quoted in the paper (presumably from the paper* by Charles E. Gregory, Assoc. M. Am. Soc. C. E.), differs from the formula as first written by Mr. Hering, in the Report of the Baltimore Sewerage Commission, 1897, in that the exponent of A is written 0.167, whereas, as given by Mr. Hering, this exponent is 0.15. Although this difference seems slight, a comparison of results obtained by using the two forms of the formula shows that they may differ as much as 15% for large areas.

The author quotes the fact that in storm-sewer design in several large cities, widely different values of R (representing the rate of rainfall) have been used, as an indication that the earlier empirical formulas have been improperly applied. His criticism would seem

* *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 458.

Mr. Sherman.

to imply that R in these formulas was intended to represent the rate of rainfall for a storm of some particular duration. As a matter of fact, however, the use of $R = 4$, in one locality, and $R = 1$, in another, merely means that, whereas the latter designer has assumed that the critical storm for which he must make provision is one lasting an hour or more, the former has assumed a condition in which a storm lasting perhaps from 10 to 20 min. will give him the most serious conditions.

If, as Mr. Buerger says, in the application of the older formulas, "it is likely that, for a mountainous, rocky district of small size, the same constants may be used as for a flat beach town of large extent", what is to prevent a similar misuse of constants in his proposed formula?

Mr. Buerger's statement, "that formula methods are much more popular", may be true if tested by the number of individual engineers using one method or the other. On this point the speaker has no information, but he is of the opinion that present practice in the engineering departments of the larger American cities, and some of the smaller ones where considerable attention has been paid to this subject, is distinctly in favor of the use of the rational method. Inquiry of the officials of seventeen such places has shown that ten of them use the rational method or some of its modifications, two use the McMath, three the Bürkli-Ziegler, and two the Hering formula. The places referred to are as follows:

METHODS USED IN ESTIMATING STORM-WATER RUN-OFF IN VARIOUS CITIES.

The Rational Method and its Modifications.—Ten places.

New York City, Borough of Queens.
" " " " Richmond.
" " " " The Bronx.
Boston, Mass.
Baltimore, Md.
Cleveland, Ohio.
Cincinnati, Ohio.
St. Louis, Mo.
Pawtucket, R. I.
San Francisco, Cal.

The McMath Formula.—Two places.

New York City, Borough of Brooklyn.
Louisville, Ky.

The Bürkli-Ziegler Formula.—Three places.

New Orleans, La.
Worcester, Mass.
Cambridge, Mass.

Mr. Sherman.

The Hering Formula.—Two places.

New York City, Borough of Manhattan.

Newark, N. J.

The speaker also differs with Mr. Buerger's apparent assumption that the results obtained by the application of his formula may be fairly tested by comparison with the results of such sewer gaugings of storm flow as are available, because by far the greater number of these gaugings have been made when the storms were of much less magnitude than those on which design must be based. It must also be borne in mind that in design it is of the first importance that the designer form a reasonable judgment of the future condition of the district under consideration, and adopt his coefficients and constants so that the drains may be adequate when the district has developed, rather than at the present moment, except in the rare case of the district which is already fully developed. It appears, however, that in the comparisons made by the author, his formula gave figures less than the actual gaugings in a large percentage of the cases. It appears, also, that, in making his comparisons, he has omitted many reported gaugings. The Report of the Committee of the Boston Society of Civil Engineers, on Run-Off from Sewered Areas*, contains figures for many more gaugings than those quoted by Mr. Buerger.

In discussing the rational method, the author states that the most complete and satisfactory treatment of it is that given by Mr. August Frühling. Mr. Frühling's analysis is unquestionably complete, and should be studied carefully by every sewer designer. In the speaker's opinion, however, the treatment of the problem given by him is needlessly complex. According to Frühling's analysis, the coefficient, C, in the rational formula, $Q = C\ I\ A$, is made up of three parts: first, a coefficient of rainfall distribution; second, a coefficient of retardation due to the fact that under ordinary conditions all portions of the drainage area are not contributing water at the same rate; and third, a coefficient of retention or of imperviousness, depending mainly on the porosity and roughness of the surface. A clear conception of the effect of these various factors is of the greatest importance in studying the results of gaugings, particularly as many such gaugings have been made when the duration of the heavy rain has been less than the time of concentration of the drainage area, so that the coefficient of retardation is an important factor. In design, however, it seems to the speaker that the most severe conditions must be assumed, in which case the coefficients of rainfall distribution and of retardation are unity, and the run-off coefficient is equal to the coefficient of imperviousness.

The proposed formula seems to be open to serious criticism as to the method in which the rainfall factor is introduced. Mr. Buerger

*Journal, Boston Soc. Civ. Engrs., June, 1914.

Mr. Sherman.

apparently implies that records of intense storms, lasting 60 min., may often be obtained when trustworthy records of intensity of precipitation for shorter periods are not to be had. As a matter of fact, the reverse is more likely to be the case. Intense storms of short duration are much more frequent than those of longer duration, and in case the rainfall record covers but a limited period, it is much more probable that it includes figures approximating maximum or critical conditions for periods of less than ½ hour than for longer periods. It is extremely unsafe, therefore, to infer from any limited record of rainfalls for periods of 1 hour what the intensity of precipitation may be which should govern storm-sewer design.

Even were it possible to determine readily what rate of precipitation for a period of 60 min. is to be expected or assumed as the critical rate, for the purposes of design, it would still be improper to assume that the intensity for any other period of time bears some definite ratio to that for 60 min., as seems to be the case in the proposed formula. The only statement in the paper relative to this ratio, which the speaker has been able to discover, is that at the foot of page 1146, where it is assumed that "$I = \frac{80\,K}{20 + t}$", or, in other terms, $\frac{I}{K} = \frac{80}{20 + t}$. This relation holds true only for a rainfall curve of the form $I = \frac{A}{20 + t}$. Although this curve has been assumed by some designers as representing approximately rainfall conditions in the Eastern Atlantic States, others have derived altogether different curves, and this form would not be at all applicable to some other parts of the country.

The speaker has had occasion recently to study rainfall records in a number of the more important cities throughout the United States, and to bring together rainfall curves suggested by various engineers for different localities. Though the form indicated by the formula, $I = \frac{A}{B + t}$, is popular with a large number of engineers, for some localities values of B widely differing from 20 have been found necessary to fit the conditions discovered, and other engineers have used formulas of the exponential type, represented by the equation, $I = \frac{D}{t^x}$, or even more complex forms of expression.

Fig. 6 shows a series of curves representing the ratios between the intensity of precipitation for any period and the precipitation for a period of 60 min. The full line represents the curve assumed by Mr. Buerger, $\frac{I}{K} = \frac{A}{20 + t}$. The other curves represent the ratios which would be obtained from rainfall curves proposed by several engineers

for various localities. It is seen that, although Mr. Buerger's curve follows, in general, near the median line, the variations are very considerable, so that, if the rainfall curves presented for the other localities are even approximately correct, material errors might be introduced by the use of the equation adopted by Mr. Buerger. Mr. Sherman.

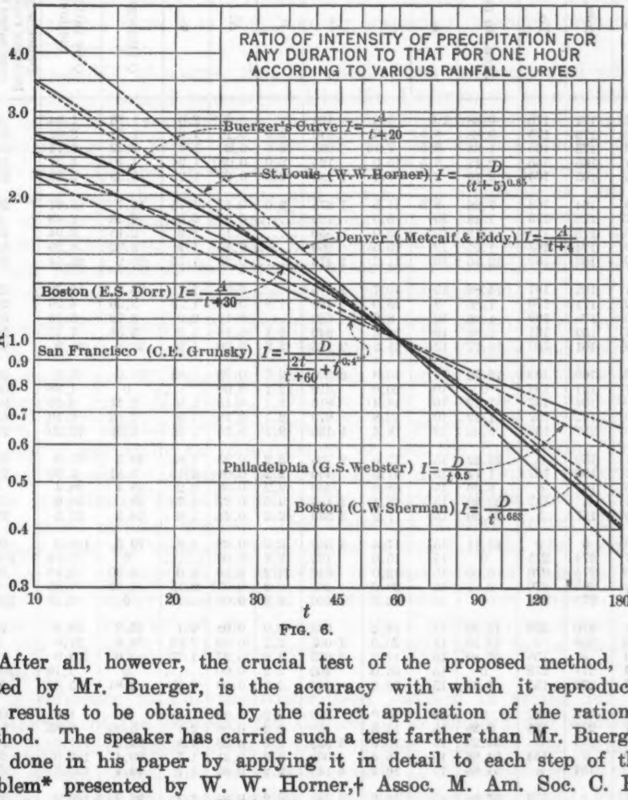

FIG. 6.

After all, however, the crucial test of the proposed method, as stated by Mr. Buerger, is the accuracy with which it reproduces the results to be obtained by the direct application of the rational method. The speaker has carried such a test farther than Mr. Buerger has done in his paper by applying it in detail to each step of the problem* presented by W. W. Horner,† Assoc. M. Am. Soc. C. E., with the results given in Table 16.

From Table 16 it is at once apparent that only in one case of the forty-two computations tabulated does the result obtained by Mr. Buerger's method agree with that computed by the rational method. In every other case Mr. Buerger's method gives a less result, and

* As recomputed in detail for Metcalf and Eddy's "American Sewerage Practice."

† *Engineering News*, Sept. 29th, 1910.

Mr. Sherman.

TABLE 16.—COMPARISON OF RESULTS OBTAINED BY BUERGER'S FORMULA WITH THOSE OBTAINED BY THE RATIONAL METHOD, FOR A SEWER DISTRICT IN ST. LOUIS. C ASSUMED AS 0.50; $K = 1.61$.

No.	From.	To.	Area, in acres.	*m.*	*s.*	*L.*	*P.*	*N.*	*q.*	*Q* (Buerger).	*Q* by rational method.	Deficiency of Buerger result as compared with rational method—Percentage.
1	177	176	0.88	10	5.0	125	......	0.09?	2.0	1.76	1.94	9.3
2	176	175	0.88	8.5	5.0	540	2.0	0.15	1.8	1.58	3.62	56.2
3	175	165	1.63	8.5	7.15	950	2.0	0.20	1.7	2.77	6.71	58.6
4	166	165	2.12	8.5	15.0	125	2.0	0.05?	1.94	4.10	4.67	12.2
5	165	164	3.75	8.5	7.4	1 180	2.0	0.20	1.7	6.39	9.39	31.9
6	164	163	7.26	8.5	11.5	1 275	2.0	0.17	1.75	12.7	15.97	20.7
7	172	173	1.13	10	5.0	125	......	0.09?	2.0	2.26	2.49	9.2
8	173	174	1.13	10	5.0	550	......	0.15	1.9	2.15	3.94	45.5
9	174	163	1.75	10	15.5	950	......	0.15	1.9	3.32	6.89	51.8
10	163	162	12.10	10	11.2	1 475	......	0.25	1.75	21.2	26.64	20.2
11	162	161	14.86	10	10.7	1 670	......	0.27	1.7	25.2	32.70	22.9
12	171	170	1.86	3	15.0	120	1.8	0.04?	1.75	3.26	4.09	20.6
13	170	169	1.86	10	8.9	630	......	0.12	2.0	3.72	5.75	35.3
14	169	161	1.86	10	12.6	940	2.1	0.18	1.9	3.53	7.40	52.3
15	161	160	16.72	12	15.2	2 105	2.2	0.20	1.9	31.8	41.2	22.8
16	160	159	16.72	14	16.8	2 520	2.2	0.25	1.85	31.0	43.6	28.8
17	131	194	0	10	20.0	310	2.1	0.00	......	0	1.88	100.0
18	194	197	1.12	10	11.4	810	2.1	0.15	1.9	2.13	6.10	65.0
19	197	257	1.12	10	12.8	970	2.1	0.18	1.9	2.13	6.10	65.0
20	257	159	5.02	10	18.2	1 135	2.1	0.15	1.9	9.55	12.20	21.7
21	159	*a*	26.22	15	17.0	2 705	2.2	0.25	1.85	48.5	61.8	21.5
22	168	167	1.77	15	47.5	440	2.2	0.04	2.15	3.81	5.20	26.7
23	167	*a*	8.36	15	68.0	480	2.2	0.03	2.2	18.4	18.4	0
24	*a*	157	30.30	15	18.1	3 105	2.2	0.27	1.75	53.0	70.9	25.3
25	157	*b*	30.30	15	17.9	3 330	2.2	0.28	1.8	54.5	71.5	23.8
26	*b*	*c*	44.11	15	17.4	3 550	2.2	0.28	1.8	79.5	100.0	20.5
27	274	275	0	15	15.0	405	2.2	0.00	......	0	1.19	100.0
28	275	270	3.00	15	12.0	840	2.2	0.13	2.0	6.00	8.98	33.2
29	265	270	0.87	15	20.0	165	2.2	0.05?	2.15	1.87	1.95	4.1
30	276	270	0	15	12.5	420	2.2	0.00	......	0	2.46	100.0
31	270	269	12.02	15	18.2	965	2.2	0.08	2.1	25.3	28.9	12.4
32	269	*c*	13.93	15	20.0	1 005	2.2	0.09	2.05	28.6	31.6	9.5
33	*c*	155	46.89	15	17.2	3 675	2.2	0.27	1.75	82.0	104.0	21.2
34	271	272	0	15	25.0	400	2.2	0.00	......	0	1.19	100.0
35	272	155	1.38	15	36.8	840	2.2	0.09	2.05	2.84	5.36	47.0
36	273	277	0	15	15.0	310	2.2	0.00	......	0	1.81	100.0
37	277	155	2.39	15	13.3	460	2.2	0.08	2.1	5.03	5.85	14.0
38	155	*d*	52.29	15	16.9	3 825	2.2	0.28	1.8	94.2	117.0	19.5
39	*d*	154	54.86	15	16.6	4 000	2.2	0.31	1.7	93.4	122.0	23.4
40	154	*e*	54.86	15	16.4	4 140	2.2	0.32	1.7	93.4	122.6	23.8
41	*e*	153	57.58	15	16.0	4 440	2.2	0.33	1.7	98.2	126.0	22.0
42	153	136	59.21	15	15.7	4 600	2.2	0.34	1.7	100.6	128.6	21.8

usually a considerably less result. In five cases the Buerger method shows a deficiency of 100%, or indicates that there would be no flow, whereas the rational method indicates an appreciable quantity of water to care for. These cases are at the upper ends of the sewers, where there is no drainage area tributary to the street inlets, but

where allowance is made in the rational method for the probability of roof-water drains being connected with the sewers. In such a case, the formula method necessarily fails, and it is necessary to apply judgment, that is to say, the rational method. Neglecting the cases where the deficiency as tabulated is 100%, the deficiency in the quantities computed by Mr. Buerger's, as compared with the rational method, ranges from 0 to 65%, and by inspection averages probably at least 25 per cent. Mr. Sherman.

It is possible, of course, that the speaker has misinterpreted some portion of the method recommended by Mr. Buerger, or he may have made mistakes in reading from the complex and small-scale diagrams accompanying the paper. He can only say that he has studied the paper carefully and endeavored to apply the method correctly, in accordance with the description, and to read the diagrams as closely as is ordinarily feasible in handling diagrams of this character. As a result of this test, the speaker is forced to the conclusion that Mr. Buerger has failed in his attempt to obtain a formula which will give results approximating those of the rational method.

ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—The writer agrees with Mr. Buerger that the McMath, Bürkli-Ziegler, and similar storm-water run-off formulas have been given practical application far beyond justifiable limits. On the other hand, he is not convinced, from anything which appears in the paper, that the Buerger formula is essentially an improvement, or that the Engineering Profession, if properly informed on the subject, does or would prefer to use a complicated formula such as the author presents, rather than the so-called rational method. This statement is based principally on the fact that the method of derivation of the formula, the specific data, or the assumptions on which it is based, are not given in the paper. The writer feels obliged to state that, in his opinion, the author is in duty bound, in fairness to the Profession, to present in full the derivation of his formula, if he expects it to be adopted generally. Mr. Horton.

In reference to this the writer feels justified in asking for an explanation of the necessity for the use of so complex and awkward a formula—a veritable *cheval de frise,* in fact. The formula contains some sixteen terms, and, if it was capable of direct solution, would require at least an equal number of mathematical operations, but it is not only an implicit formula incapable of direct solution, but even when the necessary factors (P and N) are given, it is only capable of solution by the use of a complicated diagram or by process of trial and error. The writer hesitates to accept the proposition that engineers would prefer to work out the run-off from a system of

* Albany, N. Y.

Mr. Horton. diagrams requiring such complicated operations as are necessary even for the graphical solution of this formula, rather than use the rational run-off method in which direct calculations can be applied.

The formula, it appears, involves only six factors: the length of the sewer, the run-off ratio, the tributary area, the slope of the sewer, the slope of the ground surface, and the rainfall rate. After all, the most important elements governing the results are left to judgment, as in the case of the older formulas; and, furthermore, it is difficult to see just why so thorny and centipedal an expression is necessary to express the relations between these elements. In the hope that the formula is really as good and useful as its author apparently believes it to be, the writer feels amply justified in calling for full explanation of the assumptions on which it is based and the method of its derivation, and, furthermore, for the presentation of adequate reasons for the use of so complicated an expression.

The case is by no means parallel to that of the use of the Kutter formula. The latter is in reality a much simpler expression, although so complicated that its direct solution is seldom undertaken, but it did not profit by its complexity. As a matter of fact, it seems to have been well established that the original and simpler formula, derived without taking into account the Mississippi River experiments—later proved to have been erroneous—would have been far better; but, assuming, as is no doubt the case, that the only use of either formula would generally be through the medium of graphical solution, the Kutter formula can be solved by a single operation, with Roe's diagram, and by only two operations, with Kennison's diagram. The solution of the Buerger formula, on the other hand, requires ten operations on three diagrams. Undoubtedly, these diagrams, if on a large enough scale, could be read with accuracy, but their use seems to the writer to be very tedious and the result somewhat uncertain on account of the danger of accumulated error in reading from one intersection point to another, where there are several operations to be performed on the same diagram.

Mr. Grunsky. C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—As the result of an interesting analysis of available run-off data, the author has worked out an approximation formula which may turn out to be better than the older formulas of a similar nature, such as those by Bürkli-Ziegler and McMath.

In this new formula, however, as in all others for the determination of the maximum run-off, the element of greatest uncertainty is the factor based on the relative imperviousness of the surface of the water-shed. This being the case, and all approximations of this factor being largely dependent on the judgment of the engineer, the question

*San Francisco, Cal.

arises: Why should it seem necessary to substitute a complicated formula for such simple expressions, having a rational and theoretic basis, as those presented by the writer in his paper on "The Sewer System of San Francisco and a Solution of the Storm-Water Flow Problem"?* Mr. Grunsky.

Using Mr. Buerger's notation,

$$q = \frac{4.36\, C\, K}{\dfrac{2\, t}{t + 60} + t^{0.4}}$$

is the formula for the maximum run-off, in cubic feet per second per acre, if the San Francisco type of limiting rain intensity curve is adopted, and

$$q = \frac{5\, C\, K}{\sqrt{t}}.$$

in case the parabolic form of limiting curve is preferred.

For comparison, results computed by these formulas, and containing the same elements as those used by the author, are presented in Table 17, which is the author's Table 13, with two new columns added.

TABLE 17.—COMPARISON OF VARIOUS FORMULAS FOR STORM-WATER FLOW.

All elements as assumed by the author for Table 13.

$S = 4.$

Area, in acres.	q = MAXIMUM RUN-OFF, IN CUBIC FEET PER SECOND PER ACRE.				
	Bürkli-Ziegler.	McMath.	Buerger.	Grunsky.	
				Parabola type, $C_1 = 7.75$.	San Francisco type, $C = 6.14$.
(1)	(2)	(3)	(4)	(5)	(6)
10	1.42	1.27	1.00	0.88	0.85
50	0.95	0.92	0.85	0.69	0.70
100	0.80	0.80	0.80	0.63	0.63
200	0.67	0.70	0.78	0.57	0.59
500	0.54	0.58	0.68	0.47	0.50
800	0.48	0.53	0.62	0.45	0.47
1 000	0.45	0.50	0.59	0.42	0.45
2 000	0.38	0.44	0.54	0.38	0.41
5 000	0.30	0.36	0.46	0.32	0.35
8 000	0.27	0.33	0.41	0.30	0.34
10 000	0.25	0.32	0.38	0.29	0.32

The critical time, t, in the absence of any information relating to the shape of the water-sheds, was determined from the length of the conduits as assumed by the author and from velocities of concentration, ranging from 3 ft. per sec. in the smallest areas to 6 ft.

* *Transactions*, Am. Soc. C. E., Vol. LXV, p. 294.

Mr. Grunsky. per sec. in the largest ones, noted in the table. To this 5 min. were added for the time required to reach the sewer inlets. It was assumed that the critical time, t, would be about two-thirds of that required for water to flow to the gauging point from the most remote parts of each area. This method of approximating the critical time is not advocated as being a satisfactory one, but is here adopted merely to obtain relatively consistent results for comparison with those obtained by other formulas.

The successful application of the formulas suggested by the writer depends on two things: First, the determination of the limiting rain curve for the region under consideration; and second, the approximation of a value for t. Neither of these presents any serious difficulty, one being readily deduced from rain records and the other from the topographic features of the area for which maximum discharge is to be estimated. It is not believed that the type of formula for maximum run-off which these older formulas represent, can be materially improved.

The author accepts for rain intensity the formula:

$$I = \frac{80\,K}{20 + t},$$

this being of the type which has found acceptance by many engineers in the East.

The writer, on the other hand, has suggested

$$I = \frac{C}{\frac{2\,t}{t + 60} + t^{0.4}},$$

or

$$I = \frac{C_1}{\sqrt{t}},$$

where C and C_1 are constants to be determined for any locality from the rain records.

The Eastern type of formula has the defect that it cannot be used for large values of t, etc. To illustrate this fact, the following comparison is made:

For $K = 1.00$ in., the Eastern formula, as adopted by the author, would make

$$I = \frac{80}{20 + t}.$$

The writer's San Francisco formula would make

$$I = \frac{6.14}{\frac{2\,t}{t + 60} + t^{0.4}}.$$

The writer's parabolic formula would make

Mr. Grunsky.

$$I = \frac{7.75}{\sqrt{t}}.$$

The quantity of rain in t minutes, according to these several formulas, would be, for the special case of $K = 1.00$:

By the author's Eastern formula.................................. $\frac{1.33\ t}{20 + t}$.

By the writer's San Francisco formula........ $\frac{t}{60}\left(\frac{6.14}{\frac{2\ t}{t + 60} + t^{0.4}}\right)$.

By the writer's parabolic formula.................................. $\frac{\sqrt{t}}{7.75}$.

TABLE 18.—COMPARISON OF VARIOUS FORMULAS FOR MAXIMUM RAIN INTENSITY.

The total quantity of rain in various time periods as determined by various formulas.

Time, in minutes.	INCHES PER HOUR. Buerger.	INCHES PER HOUR. Grunsky. San Francisco.	INCHES PER HOUR. Grunsky. Parabola.	TOTAL QUANTITY OF RAIN. Buerger.	TOTAL QUANTITY OF RAIN. Grunsky. San Francisco.	TOTAL QUANTITY OF RAIN. Grunsky. Parabola.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
5	3.20	3.00	3.46	0.27	0.25	0.29
10	2.67	2.19	2.45	0.45	0.37	0.41
30	1.60	1.34	1.41	0.80	0.67	0.71
60	1.00	1.00	1.00	1.00	1.00	1.00
120	0.57	0.75	0.71	1.14	1.50	1.41
240	0.31	0.63	0.50	1.23	2.52	2.00
600	0.13	0.42	0.32	1.29	4.20	3.16
1 440	0.055	0.30	0.20	1.32	7.20	4.90

Table 18 gives a comparison of the rain intensities and quantities of rain, as determined by these expressions. It will be noticed that, although there is a sufficiently close agreement between them for time periods of less than 1 hour—at which point, on the assumption of $K = 1.00$, they are made to agree—the departure of the results by the Eastern formula from those obtained by the formulas recommended by the writer becomes very marked when the value of t is materially greater than 60 min. The constants for the writer's formulas can be determined when the maximum quantity of rain for any time period up to 24 hours, or even longer, is known. This is not the case for the Eastern type of intensity formula, the use of which, for this reason, should be discouraged.

Mr. Grunsky.

A mere inspection of Table 18 will show that the quantities of rain computed by the formulas, $I = \frac{80}{20 + t}$ and $\frac{I\,t}{60} = \frac{1.33\,t}{20 + t}$, fall far short of what is known to be the fact. In any locality in which the rainfall in 1 hour is 1.00 in., the greatest possible rainfall in a day will certainly exceed 1.32 in.

Either of the two formulas for rain intensity suggested by the writer* is good for low values of t, and is also applicable to much longer periods of time than ordinarily come under consideration in urban or suburban run-off studies.

At Kansas City, Mo., during the night of September 6th and the morning of September 7th, 1914, the following quantities of rain fell:

Greatest quantity in	5 min	0.64 in.
" " "	10 "	1.01 "
" " "	15 "	1.26 "
" " "	30 "	1.56 "
" " "	1 hour	1.97 "
" " "	2 hours.....................	2.78 "
" " "	5 "	4.76 "
" " "	10 "	6.94 "

Accepting for purposes of comparison the greatest recorded rainfall of this storm during 1 hour as the value of K, that is $K = 1.97$, then the additional comparison in Table 19 can be made, which again indicates the superiority of the writer's formula (of either the San Francisco or the parabola type) when the duration of the rain is materially greater than 1 hour.

TABLE 19.—COMPARISON OF THE RAINFALL AT KANSAS CITY, MO., WITH RESULTS BY FORMULAS.

Time, in minutes.	For $K = 1.97$. Greatest Quantities of Rain, in Inches.				Quantity of Rain.
	Observed at Kansas City.	Buerger.	Grunsky, San Francisco $C = 12.1$.	Grunsky, Parabola $C = 15.3$.	Grunsky, Parabola $C = 18.0$.
(1)	(2)	(3)	(4)	(5)	(6)
5	0.64	0.53	0.30	0.57	0.67
10	1.01	0.88	0.72	0.80	0.95
15	1.26	1.13	0.90	0.98	1.16
30	1.56	1.58	1.32	1.39	1.64
60	1.97	1.97	1.97	1.97	2.33
120	2.78	2.26	2.98	2.78	3.29
300	4.76	2.46	5.28	4.40	5.20
600	6.94	2.54	8.23	6.22	7.35

* *Transactions*, Am. Soc. C. E., Vol. LXV, p. 422.

Mr. Grunsky.

It is probable that the actual limiting rain curve for Kansas City for low values of t will fall a little beyond either of the curves indicated by the figures in Columns 4 and 5. If the rainfall records of this September storm were to be made the guide in establishing a limiting curve for short time periods, the constants in the formulas, in other words, would be determined from the rainfall in 15 and 30 min. rather than from the fall in 60 min.

For a limiting rain intensity curve determined from such a single storm record as that in September, 1914, at Kansas City, the writer would recommend that C_1 be introduced into the parabolic formula at a value of 18. That is, for such a special case, there would be:

$$I = \frac{18}{\sqrt{t}}.$$

It may be added, in conclusion, that the simplicity of its form and close correspondence with the observed facts, both for short and for fairly long time periods, are elements in favor of the use of the parabolic type of limiting rain curve.

Mr. Bleich.

SAMUEL D. BLEICH,* ESQ. (by letter).—Mr. Buerger's new run-off formula again calls attention to the uncertainties and general confusion surrounding the technique for determining the run-off from the valuation of the numerous elements on which it depends. Almost every conceivable physical feature of a drainage area affects the run-off in some manner, but the relation of each can only be described qualitatively. These features have not as yet been adequately expressed mathematically.

Mr. Buerger gives a résumé of the various empirical formulas which have been proposed for the computation of run-off in terms of the salient characteristics of the drainage areas, such as size, general slope, relative imperviousness, and the length of the sewer. These formulas, it is claimed, are defective and give inconsistent results, as they were devised to fit local conditions. His objection to the so-called rational or "time element" method is that it is difficult of application; and he proposes his formula as its "acceptable equivalent".

The expression which Mr. Buerger gives for the run-off is necessarily complicated, as the rate of run-off per acre is made to depend implicitly on six elements or characteristics of the drainage area. In other empirical formulas, such as those by McMath, Bürkli-Ziegler, and others, the run-off is given explicitly in a simple relation, in terms of four elements only: area, slope, imperviousness of ground, and rainfall intensity. As a result of the complicated form of the proposed

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Mr. Bleich. formula, three diagrams are required in solving it, whereas, for the other formulas, one is sufficient. It would be interesting to know how the form of the new formula was devised.

The rational method, as developed in actual practice, although not as simple as the standard empirical formulas, is neither more difficult nor much more time-consuming than the proposed formula. It appeals more to reason and, on account of its simplicity and directness, meets with general favor. It is true that it has its shortcomings, but so have all the empirical formulas, and probably to a greater degree. The empirical formula is involved, and the relations of the factors are obscure; in consequence, it is not expected that it will come into extensive use. Its advantage over the simple empirical formulas is not evident and, in addition, it involves certain inconsistencies which destroy its validity.

It is believed that Table 20 and Fig. 7 will facilitate the use of the Buerger formula. Table 20 contains computations of the values of N and P, and Fig. 7 solves the expression,

$$q + q^{\frac{4}{5}} N = P,$$

by a series of straight lines. With N and P obtained from Table 20, q may be obtained from the diagram, Fig. 7, at the intersection of these values for N and P.

It is generally conceded that the larger the drainage area, other conditions being equal, the smaller will be the run-off per unit of area. This is contradicted by Mr. Buerger's formula. Examining the expression for N it is seen that N decreases as A increases; and, from the expression for q, q increases as N decreases; therefore, q increases as A increases. It is conceivable that two slightly different areas, may have the same values for m, K, S, C, and L, and for these two areas Mr. Buerger's formula will give a larger run-off per acre for the larger area, which is contrary to experience. The increase of run-off per acre with increasing areas is not large, but being in the wrong direction, it militates against the validity of the proposed formula.

Mr. Buerger's formula, therefore, appears to be contrary to reason and experience in the relation which it involves between q and A; and in this respect it certainly has no advantage over the other empirical formulas which he mentions. McMath's formula is approximately consistent, and, by a proper selection of values for the two constants, C and R, it may be made to give satisfactory results. It certainly is easier to apply than the new formula.

In practice, the writer prefers the use of the rational or "time element" method, the results to be checked systematically by McMath's formula, with values for the constants suitably selected in both cases.

Mr. Bleich.

DIAGRAM FOR THE SOLUTION OF THE PROPOSED RUN-OFF FORMULA BY MR. C.B. BUERGER.

$$q+q^{\frac{4}{5}}N=P;\quad N=\frac{L}{79\sqrt[5]{CAS^2}\;(24+\frac{16}{\sqrt{m}})};\quad P=\frac{3.33\sqrt{m}\,KC}{\sqrt{m}+0.87}.$$

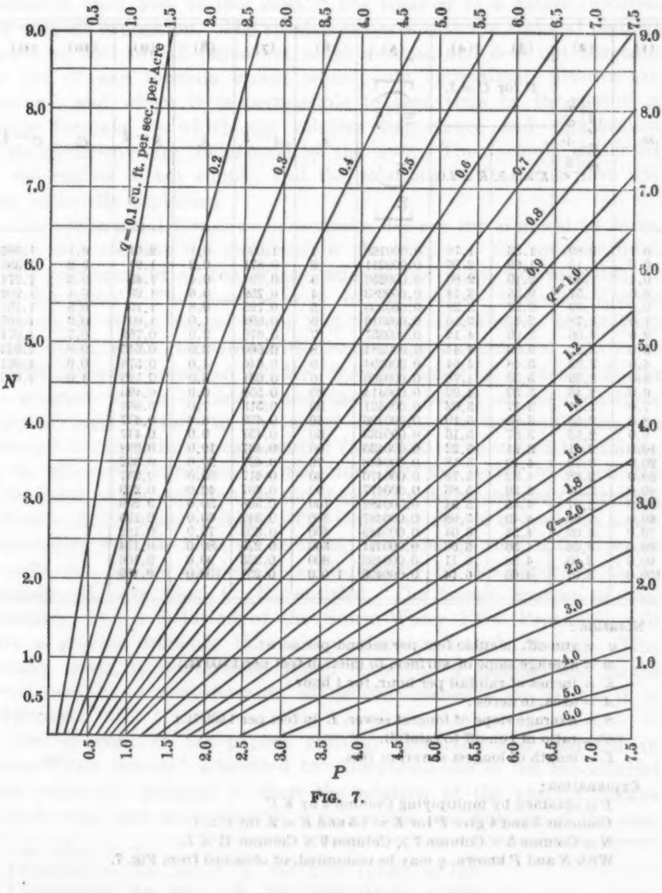

FIG. 7.

Mr. Bleich.

TABLE 20.—COMPUTATION OF VALUES OF N AND P IN THE RUN-OFF FORMULA PROPOSED BY MR. BUERGER.

$$q + q^{\frac{4}{5}} N = P ; \; N = \frac{L}{79 \sqrt[5]{C A S^2}\left(24 + \frac{16}{\sqrt{m}}\right)} ;$$

$$P = \frac{3.33 \sqrt{m}\, K\, C}{\sqrt{m} + 0.87} .$$

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
		P for $C = 1$.								
m.	$\frac{3.33 \sqrt{m}}{\sqrt{m} + 0.87}$	$K = 1.5$.	$K = 2.0$.	$\left[79\left(24 + \frac{16}{\sqrt{m}}\right)\right]^{-1}$	A.	$A^{-\frac{1}{5}}$.	S.	$S^{-\frac{2}{5}}$.	C.	$C^{-\frac{1}{5}}$.
0.1	0.89	1.33	1.78	0.000168	1	1.000	0.1	2.51	0.1	1.585
0.2	1.13	1.69	2.26	0.000211	2	0.871	0.2	1.91	0.2	1.380
0.4	1.40	2.10	2.80	0.000256	3	0.794	0.4	1.45	0.3	1.274
0.6	1.57	2.35	3.14	0.000283	4	0.759	0.6	1.23	0.4	1.202
0.8	1.69	2.53	3.38	0.000303	5	0.725	0.8	1.10	0.5	1.151
1.0	1.78	2.67	3.56	0.000316	6	0.692	1.0	1.00	0.6	1.107
2.0	2.06	3.09	4.12	0.000357	7	0.676	2.0	0.759	0.7	1.074
3.0	2.22	3.33	4.44	0.000381	8	0.660	3.0	0.646	0.8	1.045
4.0	2.32	3.48	4.64	0.000394	9	0.646	4.0	0.576	0.9	1.021
5.0	2.39	3.59	4.78	0.000406	10	0.631	5.0	0.525	1.0	1.000
6.0	2.46	3.69	4.92	0.000414	20	0.550	6.0	0.490		
7.0	2.51	3.77	5.02	0.000421	30	0.518	7.0	0.468		
8.0	2.55	3.83	5.10	0.000426	40	0.479	8.0	0.437		
9.0	2.58	3.87	5.16	0.000433	50	0.457	9.0	0.417		
10.0	2.61	3.91	5.22	0.000436	60	0.437	10.0	0.398		
20.0	2.79	4.19	5.58	0.000458	70	0.427	20.0	0.302		
30.0	2.88	4.32	5.76	0.000470	80	0.417	30.0	0.257		
40.0	2.93	4.39	5.86	0.000477	90	0.407	40.0	0.229		
50.0	2.97	4.45	5.94	0.000482	100	0.398	50.0	0.209		
60.0	2.99	4.49	5.98	0.000487	200	0.347	60.0	0.195		
70.0	3.02	4.53	6.04	0.000489	400	0.302	70.0	0.182		
80.0	3.04	4.56	6.08	0.000491	600	0.275	80.0	0.174		
90.0	3.05	4.58	6.11	0.000493	800	0.263	90.0	0.166		
100.0	3.07	4.60	6.14	0.000495	1 000	0.251	100.0	0.159		

Notation:

q = run-off, in cubic feet per second per acre;
m = average slope of surface, to inlet, in feet per 1 000 ft.;
K = inches of rainfall per hour, for 1 hour;
A = area, in acres;
S = average slope of longest sewer, L in feet per 1 000 ft.;
C = ratio of run-off to rainfall:
L = length of longest sewer, in feet.

Explanation:

P is obtained by multiplying Column 2 by K C
Columns 3 and 4 give P for $K = 1.5$ and $K = 2$, for $C = 1$.
N = Column 5 × Column 7 × Column 9 × Column 11 × L.
With N and P known, q may be computed, or obtained from Fig. 7.

Mr. Gregory.

CHARLES E. GREGORY,* ASSOC. M. AM. SOC. C. E.—The author has not only felt that storm-water run-off formulas have been popular, but he has also felt, as have many others, that the results obtained have not always been reliable, and his aim has been to devise a formula which would be as reliable and as susceptible to adaptation to varying conditions as the so-called rational method.

In order to add to the reliability and adaptability of the formula presented, the author has introduced more factors than have been commonly used, and, in this case, it has resulted in a rather involved and difficult expression. The speaker believes that the rational method should be used for all important sewer designs, and does not advocate the use of any formula except where only approximate results are required, and where it is permissible to save time by the use of a simple formula in which the relative importance and significance of its coefficients are understood by the user. The formula presented by the author is not simple, and the relations of its parts have not been rationally explained.

The exponential formulas in common use are the simplest in form, but do not account for variation in the shape of the area drained, or the arrangement of the drains, and do not allow for variation in time for entry. The possible error in using these formulas, due to variation in shape, ranges from 15% in one direction to about 25% in the other, when compared with the average watershed.

The formula presented by Mr. Buerger contains a factor, *L*, for the greatest length of drain, and uses the factor, *m*, to indicate a slope of land surface to the sewer inlets. The other factors are included in the ordinary exponential formulas. The use of the factor, *m*, to allow for variation in the time for the water to reach the sewer inlets is not sufficient, as the slope is by no means the controlling influence in the time of entry. The length of run is of quite as much importance, and, in built-up sections, the type of roof collection, as brought out by the speaker in a discussion† of the paper by C. E. Grunsky, M. Am. Soc. C. E., entitled "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem", will have a greater influence than either slope or length of run. This variable could be represented much better by a simple time factor ascertained by a consideration of all the elements by which it is influenced.

The speaker, in his paper‡ entitled "Rainfall and Run-Off in Storm-Water Sewers", presented two formulas, one in the exponential form rationally deduced to show the relation of the various factors to each other and make possible a reasonable modification of the con-

* Mt. Kisco, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 390.

‡ *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 458.

Mr. Gregory. stants, and one for use in connection with the rational method to get a first approximation. Under average conditions, this formula gives accurate results, as every factor in it may be directly modified to fit exactly all the assumptions which might be made in the rational method, except the expression for average velocity, which was necessarily based on average conditions, and is expressed in terms of area and average slope. The value of this average velocity, however, can be very easily and quickly tested and corrected, as soon as the first approximate result has been obtained, by using a Crane slide-rule, as modified by the speaker to give velocities directly. The basis of this formula is the ordinary rainfall intensity curve in the form:

$$I = \frac{B}{t + D}$$

in which I = the rate of rainfall, in inches per hour, B and D are empirical constants, and t = the duration of downfall, in minutes. In the original form used, $B = 105$, $D = 15$, and $I = \frac{105}{t + 15}$. By changing the constants, B and D, this curve can be made to fit almost any rainfall curve desired. Assume that $t = \frac{L}{V}$, that L is the greatest length of drain, and V is the average velocity of flow; express V in terms of A and S, add to D a time for entry, in minutes; apply a coefficient, C; assume that inches per hour of rain are equivalent to cubic feet per second per acre, Q, and the rainfall curve becomes a rational run-off formula with all its elements easily understood and in a much simpler form than the one presented. If it is desired to use the intensity curve for New York City which gives intensities which will probably occur not oftener than once in 10 years, on an average, B will = 150 and D will = 15; allow 5 min. for the rain to reach the inlets, and the entire run-off formula for such assumptions will be as follows:

$$Q = C \frac{150}{\left[\frac{L}{84 \sqrt[5]{A S^2}} + 5\right] + 15}.$$

The coefficient, C, representing the ratio of rate of run-off to rainfall, in this or any of the exponential formulas, may be assumed to be influenced most by the percentage of imperviousness, as is done by Mr. Buerger where he says the theoretical range is from 0 to 1.00, corresponding with the imperviousness of the soil, but such an assumption will not give results even approximately in agreement with gaugings. His assumptions for what he calls "less unusual conditions" ranging from 5 to 50%, as a function of population, are more nearly correct for rain periods not longer than the time for concentration.

This factor, C, is not, as stated by the author, the ratio between the run-off and total rainfall, but is the ratio between the maximum rate of run-off and the average rate of rainfall for the time of concentration, and is governed by a combination of influences which may be shown in their relation by the following formula: Mr. Gregory.

$$C = e\,(f\,g + (1 - f)\,h\,g) = e\,g\,(f\,(1 - h) + h);$$

e = coefficient of distribution which is a function of area;

f = percentage of impervious area;

g = ratio of run-off to rainfall due to storage on catchment surfaces and in the drainage structures; it is a function of t, and is influenced by the time for concentration and the relative area and character of the time zones;

h = ratio of run-off to rainfall due to perviousness of soil; it is a function of t and I.

The speaker, in a previous paper, deduced empirical values for C in terms of t. The value for C, as deduced from gaugings in the Sixth Avenue sewer in New York City, may be obtained by assigning values in the above formula as follows:

$$e = 0.9,\ f = 0.9,\ g = 0.5,\ \text{and}\ h = 0.10;$$

then $$C = 0.9 \times 0.5\ (0.9\ (1 - 0.10) + 0.10) = 0.41.$$

Additional data are being collected by the speaker, the better to establish values for e, g, and h. The factors, e and g, vary in opposite directions, tending to balance each other, and, for ordinary and similar conditions, a constant value for C may be used for maximum results without very serious error, after it has been established by giving due weight to its various factors. Assumed relations between population and impervious area should be used with great care, and should not be expected to give accurate results.

The use of diagrams to give results for various factors of a formula is to be commended, and the values of e, g, and h can be expressed much better in this way, as the law of variation for each changes at various points.

Studies have been made by the speaker to show the relation between the size of the sewer and the amount of damage done by flooding at various time intervals. Curves showing the relation between cost of sewers of various capacities, expressed in terms of frequency of flooding, to corresponding damage costs indicate that for certain cheap developments it would be more economical to have no sewer at all; though, at the other extreme of city development, similar to parts of the Borough of Manhattan, New York City, one flood would cost far more in damages than a sewer large enough to take the maximum storm. An important factor of sanitation is convenience and

Mr. Gregory. suitability, which must be counted in favor of a sewer, or a larger sewer, and on this it is difficult to place a financial value. It is not wholly a problem of finance.

The curve given for New York on the diagram of maximum rates for 1 hour, Plate XVI, does not agree with data for the continuous 45-year record at the Arsenal in Central Park. This record gives values almost identical with the St. Louis curve shown on the same diagram.

In his discussion, Mr. Sherman gives Manhattan, New York City, as a place where the Hering formula is used. This is no longer true, as the rational method now prevails in that Borough.

Mr. Greeley. SAMUEL A. GREELEY,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read with much interest this latest contribution to the important subject of storm-water run-off. The paper shows evidence of a large amount of work, as most of the available sewer gaugings are tabulated and checked by Mr. Buerger's formula.

The author has made a very fair statement of the various formula methods and the rational method for computing storm-water run-off in accordance with present practice, and it is his conclusion that the rational method has not received the widest use because it is laborious and requires a material exercise of judgment. This would indicate that two elements of value in the formula method are simplicity and speed of computation, and the elimination of the use of judgment. The writer agrees that the feature of chief value in that method is simplicity. He believes, however, that the exercise of judgment in the design of storm-water sewers should be promoted rather than discouraged. Even a formula computation will not give satisfactory results for any one locality, unless the user's judgment has been trained by experience in the use of the particular formula under existing local conditions. The Chicago run-off formulas computed by Mr. Hill,† are among the simplest in use and, with proper diagrams, permit of rapid computation. It appears to the writer that, in simplicity and rapidity, the proposed formula and the diagrams for its solution do not compare with some others now in use. It must stand, therefore, on its wider and more accurate application.

The formula proposed by Mr. Buerger, as shown in Equations 9, 10, and 11, is not a simple expression, and is not readily comprehended by engineers who have not had the opportunity to make special studies of formula derivation. The following comments, with reference to the units used in the formula, are submitted.

Coefficient, C.—It is not clear, from the definition given by the author, whether C is a ratio of the total volume of rainfall eventually reaching the sewers to the total rainfall, or the ratio of the maxi-

* Winnetka, Ill.

† *Journal*, Western Soc. of Engineers, October, 1902.

mum rate of run-off to the average rate of rainfall during the period of concentration to the sewer under consideration. The values of C, as determined in actual gaugings, represent the latter ratio. Mr. Greeley.

Slope, S.—The author defines the factor, S, as the average slope of the longest sewer, L, in feet per 1 000 ft. It is not clear whether this means the composite average of a series of sewer lines at different slopes or the average slope of the sewer between two points, irrespective of breaks in the grade. The latter interpretation of this factor would simplify the computations.

Average Slope of Ground, m.—In his formula the author introduces a factor to represent the average slope of the ground surfaces to the sewer inlets. The writer's experience has been that this factor is difficult to ascertain accurately in practice. A large sewer area covering a broken surface will show considerable variation in the average slope at different points along the line. The relative weight of these different slopes in the general average is hard to determine. In the rational method this factor is accounted for in the time allowance for the flow of storm-water from roofs, yards, etc., to the sewer inlets. This time allowance is added to the time of concentration for the sewer, and is ordinarily not more than 15% of the total concentration period and is frequently less. Errors in the assumed time of entrance into the sewer are, therefore, of less relative importance.

Rainfall per Hour, K.—The author uses a rainfall factor equal to the expected rate, in inches per hour for a period of 1 hour, and recommends that sewers be designed to receive the run-off from rains of an intensity which may be exceeded once every year. The writer believes that this is too frequent a surcharge for many cases, and that in certain more critical conditions provision should be made for a higher rate of rainfall.

A comparison of the work required to determine the values of the factors entering into Mr. Buerger's formula with similar steps required in the rational method does not appear to the writer to favor the new formula. The coefficient in both cases must be selected with judgment. In the Buerger formula, it is necessary to determine the length of the longest sewer run and its average slope. With these factors determined, it is not difficult to estimate the average velocity and resulting time of flow in the sewers. The factor, m, required in the Buerger formula is, in the writer's judgment, difficult to determine with accuracy, and is not as easy to apply as the constant time factor used with the rational method for the flow time of storm-water to the sewer inlet. The rainfall factor, K, in the Buerger formula, will be constant for any one vicinity, and, therefore, the selection of varying rates of rainfall corresponding to different times of concentration is eliminated.

Mr. Greeley.

The writer has recently designed a system of storm-water sewers for a town in the West having a population of about 5 000. The town was divided into three storm-sewer districts. A ridge about 40 ft. high runs through these districts near the center, separating two flat areas, a lower and an upper. In one of these districts it was necessary, in order to avoid excessive cuts, to carry the main sewer around the end of the ridge. On this account the length of sewer run and the time of concentration for District B were considerably greater than those for District A, although the two districts were not very different in area. The writer made the computations for storm-water run-off in accordance with the rational method and also with Mr. Hill's modified Bürkli-Ziegler formula. The coefficient used with the rational method was determined from a series of gaugings made on the combined sewers now used in the town, in accordance with Table 21, so that a very fair determination of this factor was possible. Since making these computations, the writer has computed the storm-water run-off by the Buerger formula, and was interested to find that the results compared very favorably with the determinations according to the rational method. In both computations, the smaller run-off was estimated for the larger area on account of the longer time of concentration. The results of these computations are shown in Table 22. In thus using the Buerger formula, the writer found it difficult to make an accurate estimate of the value of *m*, therefore he made two computations, using different values of *m*.

TABLE 21.—Gaugings in Cherry Street Sewer, Winnetka, Ill.

Date, 1912.	Time before maximum discharge, in minutes.	Area tributary in this time, in acres.	Inches per hour.	Cubic feet per second.	Maximum flow in sewer in excess of dry-weather flow.	C.	Remarks.
July 20th........	75	881	0.168	62.1	18.5	21.7	Sudden moderate shower.
July 20th........	50	320	0.200	64.0	15.0	23.4	12 hours after previous storm.
July 23d........	25	160	0.450	73.9	15.5	20.9	Short, sharp shower.
July 28th........	60	365	0.154	56.2	12.3	21.8	Second of two showers.
August 8th......	30	195	0.390	76.8	11.7	15.2	Short, quick storm.
August 9th......	75	381	0.22	82.3	13.0	15.8	8 hours after previous storm.
November 12th..	20	130	0.90	116.8	20.2	17.3	Sharp, short shower.

There is one point about Mr. Buerger's paper which deserves comment. He has tabulated most of the recorded gaugings of storm flows in sewers, and has compared these gaugings with results computed by his formula method. It would appear that the formula method should give the maximum run-off to be expected from the local conditions of each gauging. The gaugings, however, do not show maximum run-offs, but, in many cases, figures which are considerably below

the maximum run-off. The comparison should be judged with this fact in mind. Mr. Greeley.

TABLE 22.—COMPUTATIONS FOR STORM-WATER RUN-OFF BY DIFFERENT METHODS.

Factors.	BUERGER FORMULA. District. A	BUERGER FORMULA. District. B	BUERGER FORMULA. District. A	BUERGER FORMULA. District. B	RATIONAL METHOD. District. A	RATIONAL METHOD. District. B	HILL'S FORMULA. District. A	HILL'S FORMULA. District. B
A = Area, in acres	160	180	160	180	160	180	160	180
L = Length of longest sewer, in feet	5 600	9 100	5 600	9 100	5 600	9 100	......	......
C = A coefficient	0.20	0.20	0.20	0.20	0.20	0.20	0.55	0.55
K = Rate of rainfall, in inches per hour	0.9	0.9	0.9	0.9	1.2	0.95	1.0	1.0
S = Average slope of sewer, L, per thousand	7.8	5.2	7.8	5.2	7.8	5.2	......	......
m = Average slope of ground, per thousand	7.0	5.0	2.0	1.0	......	......	......	......
T = Time of concentration, in minutes	......	......	......	......	33	50	......	......
P	0.5	0.45	0.4	0.35	......	......	......	......
N	0.5	0.9	0.4	0.75	......	......	......	......
Q = Total run-off, in cubic feet per second	48.0	36.0	41.6	30.6	38.5	34.2	24.0	25.2

In conclusion, the writer finds that this formula gives satisfactorily accurate results as compared with the rational method for the particular application made, but he does not consider it easier to use, and he does not favor attempts to reduce the exercise of judgment in storm-sewer design. The value of m is not easily determined. It would seem to him that engineers should attempt to secure data and gaugings which could be used as guides for judgment, rather than to reduce the importance of the judgment factor. The author's discussion of run-off has added materially to the data on that subject, and his suggestions as to the frequency of surcharge and the general similarity of rainfall intensities over comparatively wide areas, have simplified the selection of rainfall rates in storm-sewer design, and thus have made easier the exercise of judgment along this line.

W. E. FULLER,* M. AM. SOC. C. E. (by letter).—The author states: Mr. W. E. Fuller.
"There is need for a formula of general application which can be used in a new locality with some reasonable assurance that it will be fairly appropriate, will be applicable for large areas or small, flat or steep slopes, and heavy or light rainfalls.

* * * * * * * * * *

"The writer has developed such an empirical formula for storm-water run-off, and it is presented herein, together with a detailed comparison with all published gaugings available to him."

In order to cover such varied conditions as these, it is essential that the relations existing between the different factors in the formula shall

* New York City.

Mr. W. E. Fuller.

be consistent. The author does not state that there is any limit to the size of the catchment areas for which his formula is applicable. His diagrams give values for his factors which would be proper for large catchment areas, and he makes comparisons of streams having catchment areas up to 100 sq. miles. It may be assumed, therefore. that he expects his formula to be applicable for such areas. The writer has studied the formula, particularly as to its application to large areas, and is convinced that the relations proposed are not true for such cases.

In order to test it, the writer has made computations by using the author's formula, the results of which are shown in Fig. 8. This is a plotting of the effect of different factors on the run-off for different catchment areas. In making these comparisons, values for the different factors were assumed, giving a somewhat wide range in order to cover the ordinary conditions which will be met. The relative effect of the factors is shown by the change in q due to changing one factor while all the other factors are kept constant, or, in the case of L and A, in proportion. For all plottings showing changes in C, S, and m, K is taken as 1, and L is taken as the square root of A (A being in square feet).

In Table 23 are given the percentages of change in q due to changing the different factors for 10, 1 000, and 100 000 acres, as taken from Fig. 8.

TABLE 23.—RELATIVE INFLUENCE OF DIFFERENT FACTORS IN MR. BUERGER'S FORMULA:

$$q + \frac{L}{79\sqrt[5]{C\,A\,S^2}\left(24 + \frac{16}{\sqrt{m}}\right)} \times q = \frac{3.33\ \sqrt{m}\,K\,C}{\sqrt{m} + 0.87}.$$

Area, in acres.	Percentage of Change in Values of q Due to Given Changes in the Various Factors.				
	L, in feet, changed from $2\sqrt{A}$ to $\sqrt{A}$.	S, changed from 5 to 50.	m, changed from 5 to 50.	K, changed from 1 to 2.	C, changed from 0.15 to 0.8.
10	8- 15	9- 12	26-29	100-115	440-490
1 000	27- 48	35- 39	19-24	105-120	500-600
100 000	70-110	100-110	12-16	115-135	630-850

It will be seen from Fig. 8 and Table 23 that the relative effect of all the factors varies for different values of A.

The effects of the factors, as shown in Table 23, are not for the extreme cases. This may be seen by an examination of Plate XVII.

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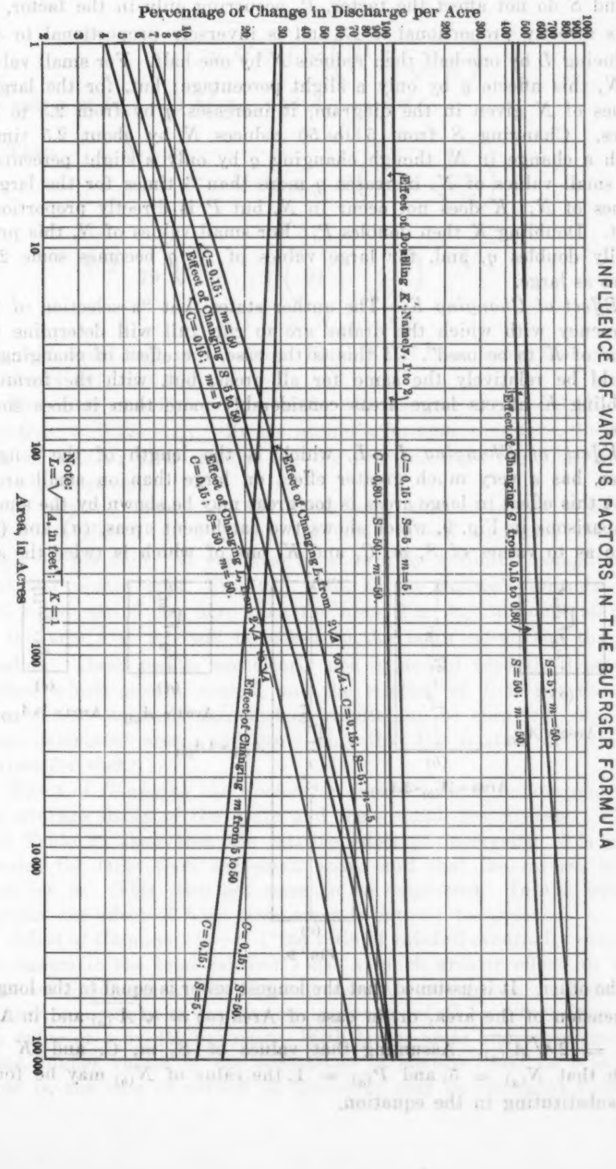

FIG. 8.

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L and S do not affect the factor, P, occurring only in the factor, N. N is directly proportional to L, and is inversely proportional to $S^{\frac{2}{5}}$. Reducing L by one-half then reduces N by one-half. For small values of N, this affects q by only a slight percentage; but, for the largest values of N given in the diagram, it increases q by from 2.3 to 2.5 times. Changing S from 5 to 50 reduces N by about 2.5 times. Such a change in N, though changing q by only a slight percentage for small values of N, increases q more than 3 times for the largest values of N. K does not occur in N, but P is directly proportional to it. Doubling K then doubles P. For small values of N, this practically doubles q, and, for large values of N, q becomes some 2.35 times as large.

Effect of Changing K.—The author states that "a selection of the frequency with which the drains are to run full will determine the value of K to be used". If this is the case, the effect of changing K should be relatively the same for all areas, but, with the formula, doubling K affects large areas considerably more than it does small ones.

Effect of Changing L.—L, which is the length of the longest sewer, has a very much greater effect on large than on small areas. That this effect in large areas is too great may be shown by the simple comparisons in Fig. 9, which shows two catchment areas, (a) and (b), alike as to values of S, m, C, and K, one of which is twice the size

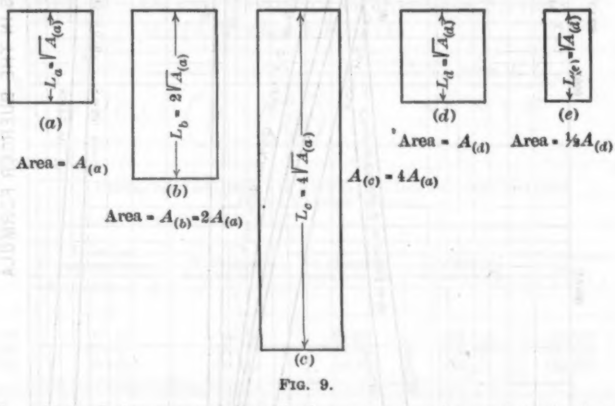

FIG. 9.

of the other. It is assumed that the longest sewer is equal to the longest dimension of the area, or, in case of Area (a) $= \sqrt{A_{(a)}}$ and in Area (b) $= 2\sqrt{A_{(a)}}$. Assuming that values of S, m, C, and K are such that $N_{(a)} = 5$, and $P_{(a)} = 1$, the value of $N_{(b)}$ may be found by substituting in the equation,

Mr. W. E. Fuller.

$$N = \frac{L}{79 \sqrt[5]{C A S^2} \left(24 + \frac{16}{\sqrt{m}}\right)}$$

the values of $L_{(b)} = 2 \sqrt{A_{(a)}}$ and $A_{(b)} = 2 A_{(a)}$.

Then $$N_{(b)} = \frac{2 \sqrt{A_{(a)}}}{79 \sqrt[5]{C 2 A_{(a)}} S^2 \left(24 + \frac{16}{\sqrt{m}}\right)},$$

or $$\frac{2}{\sqrt[5]{2}} \; \frac{\sqrt{A_{(a)}}}{79 \sqrt[5]{C A_{(a)} S^2} \left(24 + \frac{16}{\sqrt{m}}\right)} = \frac{2}{\sqrt[5]{2}} N_{(a)}$$

$$= 1.74 \times N_{(a)} = 8.7.$$

$P_{(b)}$ will equal $P_{(a)}$ or 1. From Plate XVII it is found that $q_{(a)} = 0.117$ and $q_{(b)} = 0.062$.

The total run-off from the areas will then be $Q_{(a)} = 0.117 \, A_{(a)}$ and $Q_{(b)} = 0.124 \, A_{(a)}$. By the author's formula, therefore, the addition of another area exactly like the first increases the total run-off by only 6 per cent.

It may be shown, in the same way, that, if three areas are added, as shown in Fig. $9_{(c)}$ the total run-off increases only by about 11 per cent. It can also be shown, by a similar computation, that the square area, Fig. $9_{(d)}$, will, by the author's formula, give 15% more run-off per acre than the area, Fig. $9_{(e)}$, although all parts of the area Fig. $9_{(e)}$ will be closer to the main sewer and the area smaller. These results are clearly not in accord with the true conditions which govern run-off, and the relation of L is not the true one. These obviously inconsistent results can be obtained only with large catchment areas, and prove only that the relation of L is not correct for such.

Effect of Changing the Factors, S and m.—These factors represent the average slopes of the drain and the ground, respectively. Fig. 8 and Table 23 show that the relative effect of increasing S is much greater for large than for small areas, and that the reverse is the case for m. This does not seem to be consistent. In the writer's opinion, the effect of S on large areas is entirely too great.

Effect of Changing C.—C ("the ratio of rainfall eventually reaching the sewers to the total rainfall") has a much greater effect on large than on small areas. In the extreme case which came within the writer's computations, increasing the value of C by 430% increased the value of q by 850 per cent. With larger areas and other assumptions, higher values in the percentage of change could be obtained. That is, the rate of run-off in the sewer may be increased twice as

Mr. W. E. Fuller. much as the increase of run-off from the area. To the writer this seems to be inconsistent.

The use of average slopes can only satisfy average conditions. The relation between the slopes for different parts of the basin is the real factor. Steep slopes on the drain or on the ground near the outlet may tend toward smaller run-off, rather than larger, as the bulk of the water from the near-by areas runs off before the peak of the discharge caused by the water from more remote areas is reached. Steep slopes on the remote areas will, in general, increase the run-off. For the conditions of maximum run-off, the slopes for all parts of the area must be so related as to bring as large a concentration as possible.

The effect of the slope is also dependent on other factors, such as the shape of the basin and the coefficient of run-off. Needless to say, the problem of the effect of the slope is an extremely complex one.

The rational method for obtaining the run-off by considering the different sections of the area separately, takes into account the relative slopes, and, therefore, is to be preferred to any formula based on average slope.

The author suggests that the selection of K will give the frequency with which run-off will come. The rainfall is only one of the factors involved. The conditions existing at the time the storm comes are of much importance. In a suburban or undeveloped district, the condition of the ground, whether saturated or frozen, the quantity of snow on the ground, and other factors, have a controlling effect on the run-off. The frequency is a function of the coincidental occurrence of conditions involving many factors. This frequency may or may not follow the same law as the frequency of occurrence of rainfall intensity.

The writer desires it to be understood that his criticism of the paper is limited mainly to the use of the formula for determining run-off from areas for which values of N are large. This will occur, in general, when A is large. For a more limited use, perhaps covering all but the largest of sewer areas, the formula may give results approximately correct for average conditions.

The relations existing between the factors, however, are not the true ones, and for determining run-off from very large sewer areas and from streams, the formulas, in the writer's judgment, should not be used.

Mr. G. W. Fuller. George W. Fuller,* M. Am. Soc. C. E. (by letter).—In the consideration of storm-water problems, it is a fact that the engineer is required to exercise sound judgment in several particulars. This is true whether he attempts to determine the future run-off by the so-called "rational" method, or by the Buerger or any other formula.

* New York City.

In no instance is a conclusion sounder than the premises on which it is predicated. In some cases, furthermore, the final decision ought not to be based on the ultimate maximum run-off, but should relate to the construction of drains and appurtenances which provide for all ordinary storm flows, beyond which flows the additional construction work required would not afford benefits in proportion to the cost involved. Mr. G. W. Fuller.

In the final analysis, therefore, the degree of excellence attending the design of a storm-water drain is associated largely with the question of judgment, and, to that end, Mr. Buerger's paper and the discussions thereon are helpful, in that they afford a good review of prevailing practice.

On a number of occasions, the writer has been obliged to consider probable future storm flows in connection with requirements for pumping stations and settling tanks for storm overflows, both with and without sterilization of the flow to prevent objectionable bacteria from reaching a waterway from which a public water supply is obtained. For problems of that sort, especially, it is believed that the formula proposed by Mr. Buerger has some marked advantages of convenient application, giving adequate results within reasonable limits of accuracy.

CHARLES B. BUERGER,* M. AM. SOC. C. E. (by letter).—It is rather curious that, in the voluminous discussion of this subject, so little reference has been made to the records of actual gaugings of sewer flows after storms. One might almost think that, after sewers were built, they did not actually carry off storm-water, and that the quantity they appeared to carry off was of the least importance. Mr. Buerger.

Much has been said about the form of this formula, the desirability of its use, its inconsistencies and irregularities, the limits of its possible application, and the greater desirability and convenience of other methods of determining storm-water run-off. Whether or not the results shown by its use in any way correspond with what facts show, has been ignored, and also the question as to whether other methods of determining run-off give results corresponding with actual conditions.

This is certainly attacking the problem from the wrong direction. The question which should receive the most, if not the only, weight is what is shown by the comparison of known gaugings and the results obtained by this formula and by other methods. In the paper the writer gave the first place to such investigations, and made a detailed comparison of formula results with such gaugings as were then available.

* New York City.

Mr. Buerger.

Mr. Sherman makes some slight reference to this point. He rather deprecates the making of such comparison with gaugings, because the storm flows which have been gauged are not the maximum flows for which the sewer is designed. It is true that the calculated run-off by formula from a rate of rainfall less than the maximum for which the sewer is designed will give results varying somewhat from those which would be obtained for a full sewer. The discrepancy, however, will not be very great, and the comparison is possibly as useful as if it were made for a sewer running full.

Mr. Sherman also comments:

"It must also be borne in mind that in design it is of the first importance that the designer form a reasonable judgment of the future conditions of the district under consideration, and adopt his coefficients and constants so that the drains may be adequate when the district has developed, rather than at the present moment, except in the rare case of the district which is already fully developed. It appears, however, that in the comparisons made by the author, his formulas gave figures less than the actual gaugings in a large percentage of the cases."

Whatever the future conditions may be, and whatever influence they may have on the selection of constants and the design of storm-water sewers to meet such future conditions, none of these factors can in any way affect the comparison between gaugings and calculated run-off under present conditions, using the constants for such conditions. It is certainly proper to note the fact that gaugings show at times a greater run-off than that calculated for the conditions existing at the time. Any formula which would give uniformly higher figures than the actual run-off for the same conditions, having in mind the rather erratic results obtained from gaugings, would certainly not be dependable in any way as a guide to determine probable run-off under assumed conditions. Where, for instance, the run-off under the same conditions may vary from 10 to 20 cu. ft. per sec. with a certain rainfall, it is not a fair criticism to show that a formula gives a run-off of 15 cu. ft. per sec. and that actual gaugings at times give higher results.

Attention has been called to the fact that the paper did not include any reference to the gaugings given in the report* of the Committee on Run-Off from Sewered Areas, of the Boston Society of Civil Engineers. This was not an omission; the paper was submitted in April and the gaugings were not published until June. The idea, however, that all gaugings should be used as a basis of comparison is eminently proper, and the writer submits a comparison of these gaugings with the results obtained by the formula. In selecting gaugings for comparison, only those rainfalls have been taken in which the maximum

* *Journal*, Boston Soc. of Civ. Engrs., June, 1914.

rate exceeded 1 in. per hour. Table 24 gives the results of gaugings in six places compared with the formula calculation, and Table 25 gives in detail all the rainfall records reported for the Twelfth and Diamond Street Sewer, in Philadelphia, Pa. Mr. Buerger.

Taken as a whole, the comparisons in Tables 24 and 25 show a reasonable agreement between the calculated and the gauged results.

There seems to be generally a curious misconception as to the nature of the results obtained by the so-called "rational" method. The rational method, of itself, is not such a definite one that all men can and do agree on its use and on the results obtained by it. In its application, there will usually be as many results (differing widely from each other) as the number of men using it. One man may use this method in one way and get results 25% higher than those obtained by the formula; another man may use what he calls a rational method and get results 25% lower than the formula will give. Because of this fact, any comparison of results by formula with those from so-called rational methods of calculation will be convincing only to the particular man who makes the comparison, when that particular man is satisfied—as he usually is—that his own rational results are the only ones worth considering. Another man, however, will make computations in a different way, and will be just as well satisfied that the formula errs in the opposite direction by an equal amount.

TABLE 24.—COMPARISON OF SEWER GAUGINGS WITH RESULTS OBTAINED BY FORMULA.

City, and drainage area.	Date.	A	L	K	S	m	C	q_B (Buerger).	q_G (gauging).
Hartford, Conn., Franklin Avenue at South Street (p. 348).*	July 25th, 1900.	477	8 050	1.00	2	4	0.22	0.26	0.18
Hartford, Conn., Franklin Avenue at Bond Street (p. 349).	July 11th, 1901.	263.5	6 400	1.90	3	6	0.24	0.65	0.50
Pawtucket, R. I., Newell Avenue District (pp. 335 and 355).......	Oct. 20th, 1906.	146	4 800	0.70	9	7	0.25	0.30	0.302
Newton, Mass., Hyde Brook Drainage Area (pp. 335 and 356).	Sept. 4th, 1907.	350	7 500	0.70	20	23	0.24	0.33	0.61
Newton, Mass., Hyde Brook Drainage Area (pp. 335 and 356).	Aug. 7th, 1908.	350	7 500	0.70	20	23	0.24	0.63	0.71
Wilmington, Del., Shipley Run Drainage Area (pp. 334 and 356).	July 25th, 1908.	174	5 300	1.95	30	40	0.65	3.10	3.10

* Data from *Journal*, Boston Soc. Civ. Engrs., June, 1914, pp. 334–356.

A rather interesting application of these imaginary rational results is to be found in Mr. Sherman's discussion. He points out that in five cases of his tabulation, the formula shows a deficiency of 100% where his rational method shows a flow from roof areas not connected. The formula shows a zero flow, because there is no area tributary to

Mr. Buerger.

the sewers. Mr. Sherman seems to find it possible by his rational method to obtain a run-off from roof areas where no area whatever is tributary. To the writer, it would seem that whether the area in question is a roof, a pavement, or an uncultivated field, there must still be some area for rain to fall on before it can run off and show some flow in the sewer.

TABLE 25.—COMPARISON OF GAUGINGS IN COMBINED SEWER AT TWELFTH AND DIAMOND STREETS, PHILADELPHIA, PA.*

$A = 1\,360, L = 10\,700, m = 14, S = 5.6, C = 0.65$, population $= 100\,000$.

Date.	K	P	q (Buerger).	q (Actual).	Ratio.
1903					
June 10th......	1.32	2.30	1.45	0.49	2.96
June 10th......	1.09	1.75	1.10	1.33	0.83
1906					
May 28th......	1.04	1.75	1.10	1.02	1.08
June 16th......	1.30	2.35	1.50	1.05	1.43
Aug. 2d.......	1.00	1.80	1.15	0.88	1.31
Aug. 21st......	1.00	1.80	1.15	0.94	1.22
Aug. 24th......	0.60	1.10	0.65	0.71	0.92
Oct. 5th.......	1.45	2.50	1.60	1.02	1.57
1907					
May 16th......	0.67	1.25	0.75	0.87	0.86
July 18th......	1.25	2.20	1.45	0.94	1.54
July 20th......	0.95	1.70	1.05	0.89	1.18
Sept. 28th......	0.58	1.00	0.58	0.53	1.09
1908					
May 22d.......	1.06	1.75	1.10	0.84	1.31
June 15th......	0.46	0.90	0.52	0.90	0.58
July 25th......	1.06	1.75	1.10	0.96	1.15
Aug. 25th......	0.75	1.35	0.80	0.72	1.11
Aug. 26th......	0.62	1.25	0.75	0.70	1.07
1909					
Aug. 16th......	0.52	0.95	0.55	0.56	0.98
1910					
Aug. 8th.......	0.60	1.10	0.65	0.61	1.07
Aug. 19th......	0.97	1.80	1.15	0.94	1.22
Sept. 2d.......	0.65	1.25	0.75	0.41	1.83
Sept. 6th......	1.08	1.75	1.10	1.06	1.04
Oct. 19th-20th.	0.64	1.25	0.75	0.59	1.27
1911					
June 3d........	....	....	....	....	....
July 17th......	1.00	1.80	1.15	1.09	1.05
Aug. 15th......	0.56	0.95	0.55	0.88	0.63
Aug. 30th... ..	0.69	1.30	0.80	0.88	0.91
Sept. 11th......	0.49	0.92	0.54	0.46	1.17
Nov. 6th.......	0.45	0.92	0.54	0.51	1.06
1912					
Feb. 26th......	0.95	1.80	1.15	1.01	1.14
Mar. 12th	0.75	1.35	0.80	0.81	0.99
Mar. 15th......	0.95	1.80	1.15	0.98	1.17
Nov. 1st.......	0.34	0.55	0.30	0.59	0.51
Average ratio (excluding first item)........................					1.10

* Data from *Journal*, Boston Soc. of Civ. Engrs., June, 1914, p. 357, and by letter from George S. Webster, M. Am. Soc. C. E.

A few of the members discussing the paper have devoted some attention to the question of the origin and logical form of the formula. Mr. Horner derives a similar, almost identical, formula, on certain assumptions, and points out that the formula is not of itself a rational one. Mr. Horton calls for the derivation of the formula, suggesting that its value depends in some way on this derivation and on the formula being logical. Such features do not seem to be of any importance. The formula was presented as empirical, and should stand or fall according to the accuracy of the results obtained by it. How this formula may be derived, and how logical or illogical it may be in form, seems to be entirely beside the question. Although the formula was started in its present shape and some attempt was made to formulate the existing logical relations, it was changed both in form and in constants in an attempt to make it correspond with such results of gaugings as have been made available. It was considered by the writer that logic of form should be sacrificed to accuracy of results. As it stands now, it is intended only as an empirical expression which should give results corresponding with reasonable accuracy to what will be found by the average of gaugings made. Mr. Buerger.

Mr. Weston E. Fuller deduces, as a result of a comparison of the effect of the various factors, that "the relations existing between the factors, however, are not the true ones", and suggests that the formula is not unlimited in its usefulness, but that "for a more limited use, perhaps covering all but the largest of sewer areas, the formula may give results approximately correct for average conditions". This is, in fact, the only range for which it was designed. It is intended to give reasonable results for ordinary sewer areas, and is not to be applied to the run-off from large water-sheds. For such service, it is hardly possible, in the judgment of the writer, for any formula whatever to give reasonable results, and the only guides at all worthy of credence are records of actual run-off for the particular water-shed, considered in connection with the quantity of rainfall and the probability of increased or decreased rainfall in the future.

Mr. Grunsky points out that the relation used for the reduction of rainfall to a basis of 1-hour periods is not suitable for use for such places as San Francisco and Kansas City, where the rainfall follows a different law and the total quantity of rain increases much more rapidly with the time than is obtained by the assumed relationship. This is a fair criticism, but the effect is by no means as marked as is suggested in Table 19. Within ordinary ranges, and in ordinary sewer districts from which storm-water is to be removed, a period of 2 hours for concentration may be considered practically the maximum. For such a limited period, the error of the assumed formula is not excessive. It is obvious, of course, that any formula intended for general application must be based on some median rainfall curve,

Mr. Buerger. and that those particular localities which have extremes—and San Francisco is one of these localities—will show the most marked departures from this median line. For such localities, the error of using a formula will be somewhat greater than that found in average practice. Whether such error will be excessively great depends on a number of local conditions, and these should be considered in selecting the method of determining the storm-water run-off.

Several references are made to the fact that the constant coefficient of run-off represented by C is not defined in the usual way. The writer defines this coefficient, C, as "the ratio between the run-off and the total rainfall", the more common definition being the ratio of the maximum rate of run-off to the average rate of rainfall. This change was made intentionally by the writer, and this was pointed out when he stated that "the coefficient, C, probably does not, in all cases, represent the identical conception". In selecting a value of C for use in the formula, proper consideration should be given to this definition, and the curve for C depending on the tabulation submitted by the writer is based on this revised definition.

Some exception has been taken to the writer's statement that the formula permits of work which is largely mechanical in its nature and does not require the exercise of any high degree of judgment in its application. A little examination, however, will show very plainly that this statement is a moderate expression of the facts.

Taking the various elements of the formula, it will be found that L, the length of the sewer, is always determined by topographical considerations at the first glance, and does not require much thought. In addition, although the path of the sewer may be changed somewhat, the total length will not differ materially, whatever path be assumed. C is the run-off coefficient to be determined for some future date when the district has increased in population and the run-off is greater than at the present time. The population diagram, Fig. 2, gives this coefficient, C, in terms of the future population, and thus this value requires no further thought.

The area of the district in question appears on the map, and does not need the exercise of any judgment.

The slope of the sewer is also a relatively simple matter; for all practical purposes, it is quite sufficient to take the difference of elevation between the highest point and the point under consideration, and divide this by the total length of the sewer, in thousands of feet, to get the slope per thousand. Local variations from this slope will not affect the result to such an extent as to warrant any investigation.

The value of the average slope to the sewer inlet is a little more difficult of determination; it requires the taking of the difference of elevation of the sewer inlet and the high point between sewer inlets

at a number of places within the area in question; the average of these results is the value of *m*. It may take a certain amount of labor to determine this average slope, but the work is practically all mechanical. Mr. Buerger.

The last factor of the formula, the value of *K*, can readily be derived on the assumption of the interval between full charging of the sewer and the rainfall curve, for the district in question, or for a district of somewhat similar characteristics.

A great deal of consideration has been given, and quite properly, to the question of the facility of use of this new formula and of its relative ease as compared with other methods. This, of course, is all a matter of experience in a particular use. It is natural that one unfamiliar with it would find its use cumbersome and somewhat difficult. The writer's experience suggests, however, that a reasonable amount of practice in determining values by the curves on a relatively large scale permits of very rapid and accurate work. The diagrams printed with the paper are, of course, too small to be suitable for any readings at all. The writer has found that an assistant of ordinary intelligence is easily able to work several times as rapidly with the help of suitable diagrams for this formula as with any of the ordinary variations of the rational method. What is even more marked, however, is the fact that the work is much more likely to be accurate, and is much more trustworthy.

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Paper No. 1329

THE DESIGN AND CONSTRUCTION OF FOUR REINFORCED CONCRETE VIADUCTS AT FORT WORTH, TEXAS*

BY S. W. BOWEN, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. CARL GAYLER AND S. W. BOWEN.

SYNOPSIS.

The object of this paper is to bring out a discussion of the type of reinforced concrete arch in which the reinforcement is composed of structural shapes and is designed to support the weight of the forms of plastic concrete in the arch ring on ribs.

The paper describes the design and construction of four reinforced concrete viaducts, in two of which arch spans of the above mentioned type are used. These spans range in length from 125 to 200 ft., in the clear. Three-hinged, ribbed arches are used, having hemispherical, ball and socket, cast-steel hinges. No falsework was used except such comparatively light timbering as was needed to erect the structural steel reinforcement of two of the spans.

It is concluded that, for high structures, and for those over streams subject to sudden and great variation of water level, this method is cheaper and safer than to use falsework supported from the bed of the stream.

* Presented at the meeting of October 21st, 1914.

The tables give unit and total costs, as well as the cost per linear foot, per square foot of horizontal and vertical projection, and per cubic foot of volume, for each viaduct.

INTRODUCTION.

The City of Fort Worth, in Tarrant County, Tex., is traversed by the Trinity River and its branches, the Clear and West Forks. As will be seen by the map, Fig. 1, this stream is very crooked, and cuts the town up badly, necessitating a number of bridges. The city proper lies on the high ground, around which the stream flows. The growth of the residence districts toward the east and west, and the great increase in traffic between the city proper and North Fort Worth, in which the packing houses are located, has recently made necessary the construction of a new viaduct and the reconstruction of three old ones.

The principal feature of this paper is the description of the design and construction of the three-hinged, ribbed-arch spans. In the ribs and rib braces these spans have structural steel reinforcement which is designed to support the weight of the forms and plastic concrete of the ribs and braces during construction. By this means falsework was dispensed with, except such light timbering as was used in erecting some of the structural reinforcement.

These viaducts were built with the proceeds of the sale of bonds issued by the county. The total amount of funds available for construction purposes was about $656 000, exclusive of the accrued interest on the funds in bank.

The firm of Brenneke and Fay, Consulting Engineers, of St. Louis, was selected by the Commissioners' Court to prepare plans and specifications for, and to supervise the construction of, the four viaducts.

The bridges which the viaducts were to replace were old, and too light and narrow to take care of the increasing traffic. The general feeling among the residents of the county was that the new structures should be planned for the future, and be built in such a manner, and of such materials, that they would require the minimum amount of maintenance, and last indefinitely. Therefore, reinforced concrete was selected as the material best suited to the conditions, and was used for all parts of the structures. Creosoted wood block paving

was recommended by the engineers as being light, smooth, and durable, but owing to the high cost of such paving at Fort Worth, vitrified brick was finally selected.

Design.

General Description.—The structures are known, respectively, as the Main Street, West Seventh Street, Samuels Avenue, and East Fourth Street Viaducts. They are mentioned in the order of their cost, the Main Street Viaduct being the largest of the four. The general map, Fig. 1, shows the location of the crossings, and Plate XVIII shows the four structures drawn to the same scale, so that their relative size can readily be seen.

The widths of roadways and sidewalks are as follows: Main Street, 54-ft. roadway and two 8-ft. sidewalks; West Seventh Street, 43-ft. roadway and two 6-ft. sidewalks; Samuels Avenue and East Fourth Street, 30-ft. roadway and two 5-ft. sidewalks. In each case provision is made for two electric interurban car tracks in the roadway. These tracks are placed in the center of the roadway for the Main and West Seventh Street Viaducts, and at the sides for the two smaller structures. The Main Street Viaduct is the most important of the four, and is on the main artery of travel between the city proper and North Fort Worth. Therefore, it is made wider than the others. The width of roadway is such that four wagons and two street cars can pass abreast. The width of the roadway of the West Seventh Street Viaduct is the same as the width, between curbs, of the street leading up to the crossing. This is ample to accommodate two wagons and two street cars abreast. The two smaller viaducts, being short, and of less importance, as to the volume of traffic to be carried, than the first two named, have roadways of sufficient width to allow only one wagon and two street cars to pass abreast.

From out to out of concrete construction, the length of the Main Street Viaduct is about 1 752 ft., made up of one 225-ft. arch span over the stream, two 175-ft. and one 150-ft. arch spans, one 68 ft. 9-in., two 62 ft. 6-in., seven 50-ft., and two 25-ft. girder spans. The remainder is made up of earth fills enclosed by retaining walls of the semi-gravity type. The West Seventh Street Viaduct is about 1 041 ft. long, consisting of one 137 ft. 6-in. arch span over the stream, and seven 50-ft. girder spans; the remainder consists of retaining

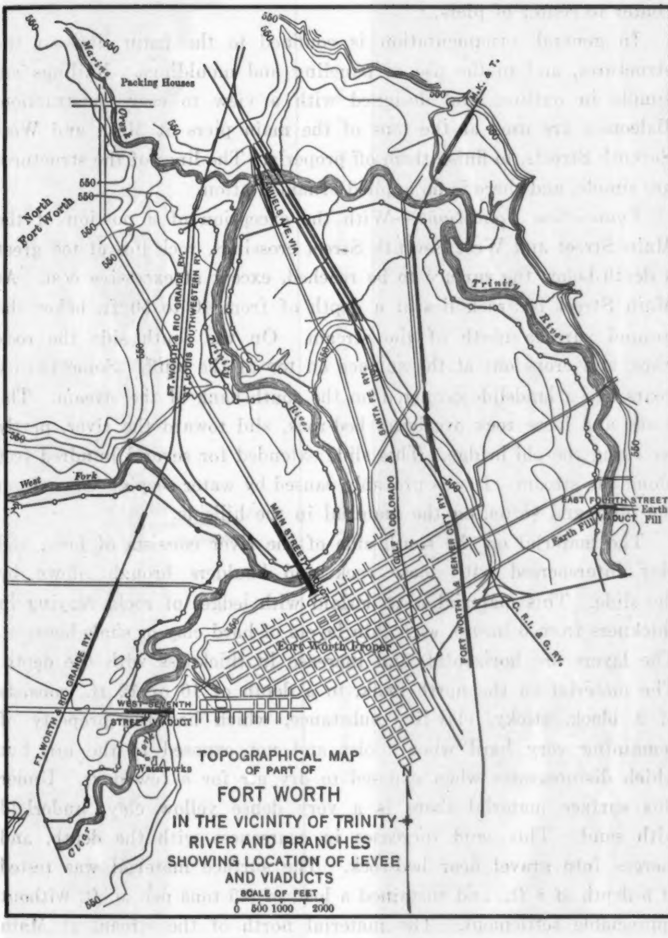

TOPOGRAPHICAL MAP
OF PART OF
FORT WORTH
IN THE VICINITY OF TRINITY
RIVER AND BRANCHES
SHOWING LOCATION OF LEVEE
AND VIADUCTS
SCALE OF FEET
0 500 1000 2000
Packing Houses
Marine Creek
North Fort Worth
West Fork
Trinity River
EAST FOURTH STREET
Earth Fill
Earth Fill VIADUCT
MAIN STREET VIADUCT
Fort Worth Proper
WEST SEVENTH STREET VIADUCT
Waterworks
Clear Fork
FT. WORTH & RIO GRANDE RY.
ST. LOUIS SOUTHWESTERN RY.
SANTA FE RY.
GULF COLORADO & SANTA FE RY.
FORT WORTH & DENVER CITY RY.
C. R. I. & G. RY.
M. K. & T.
550

FIG. 1.

walls. The other two structures each have nine 50-ft. girder spans, and are 450 ft. long, exclusive of the approaches, which consist of earth fills without retaining walls. All the spans are measured from center to center of piers.

In general, ornamentation is confined to the main lines of the structures, and to the use of paneling and mouldings. Railings are simple in outline, and designed with a view to easy construction. Balconies are used at the tops of the main piers at Main and West Seventh Streets, to finish them off properly. The lines of the structures are simple, and there is no applied ornamentation.

Foundation Conditions.—With the exception of a portion of the Main Street and West Seventh Street crossings, rock lies at too great a depth below the surface to be reached, except at excessive cost. At Main Street bed-rock lies at a depth of from 40 to 50 ft. below the ground surface north of the stream. On the south side the rock rises, and crops out at the surface on top of the bluff. Some twenty years ago a landslide occurred on the south bank of the stream. The earth and loose rock overlying bed-rock, slid toward the river, partly wrecking the old bridge. This slide extended for several hundred feet along the stream. It was probably caused by water seeping down from the bluff and softening the material in the hillside.

The material on the south side of the river consists of loam and clay interspersed with loose rock and boulders brought down by the slide. This material is underlaid with ledges of rock, varying in thickness from 6 in. up, with thin layers of hard clay or shale between. The layers are horizontal, and increase in thickness with the depth. The material on the north bank, to a depth of 10 or 12 ft., consists of a black, sticky, clay-like substance, which has the property of remaining very hard when moist and not exposed to the air, but which disintegrates when exposed to dry air for a few days. Under this surface material there is a very dense yellow clay, underlaid with sand. This sand increases in coarseness with the depth, and merges into gravel near bed-rock. The surface material was tested at a depth of 8 ft., and sustained a load of 3.3 tons per sq. ft. without appreciable settlement. The material north of the stream at Main Street is fairly typical of that at the other crossings.

River Conditions.—The Trinity River, though normally a small stream, hardly larger than a creek, is subject to sudden freshets, and

FIG. 2.—MAIN STREET VIADUCT, FORT WORTH, TEXAS, FROM AN ARTIST'S PERSPECTIVE.

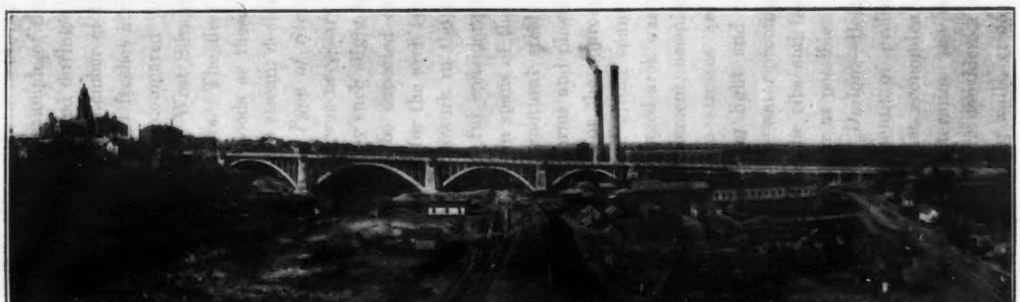


FIG. 3.—PANORAMIC VIEW OF MAIN STREET VIADUCT.

carries large quantities of drift. The banks are steep, and the run-off is rapid. As an example of the rapidity of the rises, may be mentioned a flood that occurred during the summer of 1912, before work started, and another in September of 1913, while work was under way. During the first mentioned freshet a rise of 23 ft. occurred in 24 hours. This rise was gradual compared with the last mentioned flood, where the rise, measured at West Seventh Street, amounted to a total of 16 ft. in about 1½ hours. The first 14 ft. of this rise took place in about 1 hour. Such floods as these are not at all unusual, and make work in the bed of the stream decidedly hazardous.

Selection of Type of Structure.—Because of the conditions mentioned above, it was necessary to adopt a type of structure that would not be injured by such slight unequal settlement as, under the circumstances, might be expected to occur. It was also thought advisable to use, at least for the arch spans, a method of construction that would not require falsework in the stream.

After a careful consideration of various types, it was decided to use, for the main spans of the two larger viaducts, three-hinged, ribbed arches, with structural steel reinforcement designed to support the weight of the forms and plastic concrete of the ribs and braces during construction. For the approach spans and for the river spans of the smaller viaducts, girder spans were adopted.

The three-hinged arch was selected because it would not be strained by unequal settlement, because the stresses are statically determinate, and temperature stresses are eliminated. Ribbed construction was adopted as being light and best adapted to the use of hinges, and also because no water-proofing would be required. Structural reinforcement for the ribs and braces was used in order to dispense with falsework, as far as possible.

Preliminary Design.—Before beginning the actual designing, a considerable amount of preliminary work was done in order to determine the most economical type of structure for various heights, and also to determine the economical span lengths for each type. Three types were considered, as follows: earth fills enclosed by semi-gravity retaining walls, girder spans, and arch spans.

In this preliminary work sufficient designing and laying out was done to enable the various quantities to be determined with sufficient accuracy for the purpose in view. These quantities were multiplied

by estimated unit prices, and the cost per linear foot of structure, of each type, for various heights and span lengths, was obtained. These results were then plotted, and the economical span lengths were determined, as well as the points at which it was theoretically economical to change from one type of construction to another. The information obtained from the plots was followed as closely as circumstances would permit. In some cases, however, the physical conditions encountered determined both the span lengths and the type of structure. For example, at Main Street, arch spans were used over the levee, and between the south abutment and Pier No. 1. Girder spans would have been cheaper at these points, if the height only were considered. It was not permissible to place piers in the levee, and a long span had to be used. It was not thought advisable to divide the span between the South Abutment and Pier No. 1, on account of the possibility of a recurrence of the slides mentioned previously; in other words, the fewer piers on the south bank of the stream at this point, the better. Consequently, arch spans were used for these portions of the crossing.

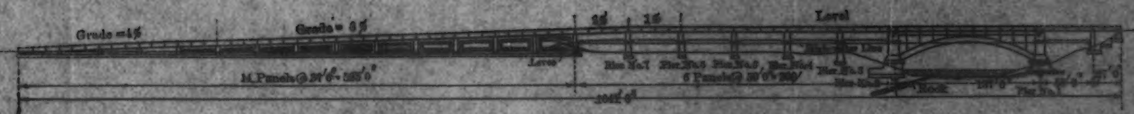
Girder spans would have been cheaper than the arch at West Seventh Street, but, as this structure is near one of the city parks, and in a high-class residence district, an arch was used over the stream, for esthetic reasons.

Girder spans were used throughout for the Samuels Avenue and East Fourth Street crossings, for the sake of economy, and to afford the maximum clear opening with minimum height. This required the use of falsework in the stream. This was undesirable, but it was considered that as in this case the girder spans were considerably cheaper than self-supporting arches, and that as these crossings were situated so that the force of the current at flood stage was much less than at the other crossings, girder spans would answer the purpose.

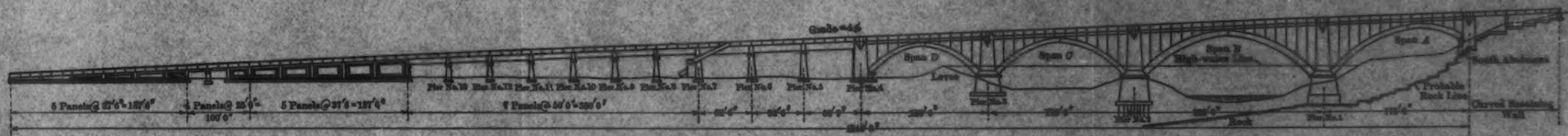
The selection of the three-hinged ribbed arch with forms supported from the reinforcement, was governed principally by the character of the foundations, and by the erratic behavior of the stream. However, at Main Street, estimates were made of the cost of this construction as compared with that of a three-hinged ribbed arch supported on falsework in the usual manner. In making this comparison, the cost of the falsework and the rib reinforcement required for this scheme was compared with that of the rib reinforcement of the arches

PLATE XVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1329.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.

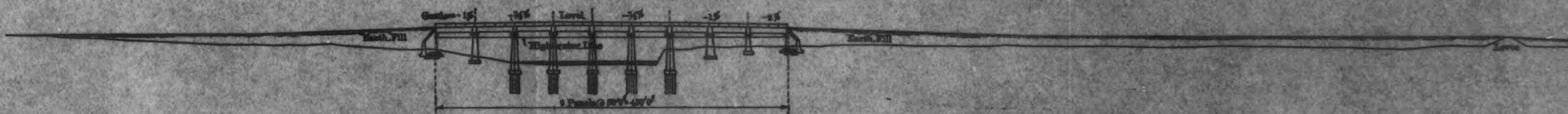
GENERAL ELEVATIONS
AND SECTIONS
FORT WORTH VIADUCTS
FORT WORTH, TEXAS



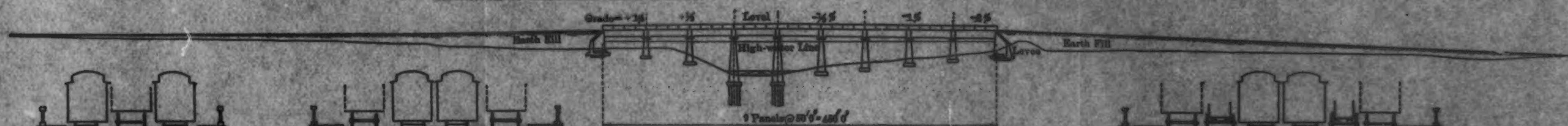
ELEVATION OF WEST SEVENTH STREET VIADUCT.



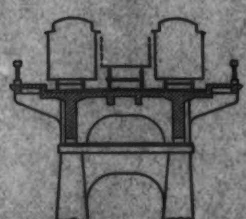
ELEVATION OF MAIN STREET VIADUCT.

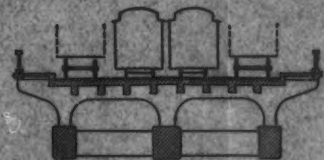


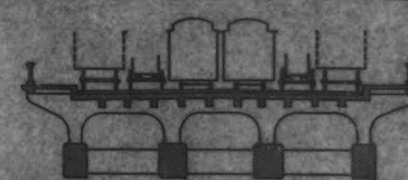
ELEVATION OF SAMUELS AVENUE VIADUCT.



ELEVATION OF EAST FOURTH STREET VIADUCT.


CROSS-SECTION THROUGH GIRDERS
SAMUELS AVE. AND EAST FOURTH ST. VIADUCTS.


CROSS-SECTION THROUGH ARCH
WEST SEVENTH STREET VIADUCT.


CROSS-SECTION THROUGH ARCH
MAIN STREET VIADUCT.

as built. The hinges, and such portions of the rib forms as were common to both schemes, were not included in the estimate.

It was found that the structure on falsework would have cost slightly less than as built. This was caused by the fact that Spans *A*, *C*, and *D* are too low for economy with the self-supporting type of construction. Span *B*, however, is high enough to make this type economical. It was decided to use the self-supporting type for all spans, because of the danger from floods, the unstable character of the soil under Span *A*, and in order to avoid the troubles due to settlement of the falsework.

The selection of the self-supporting type was amply justified by events which will be mentioned later, under the head of "Construction".

General Layouts.—After the preliminary designing had been finished, and the data obtained had been arranged for use, the work of preparing the general plans was started. The Main Street structure is typical of the work, and will be used as an example throughout the remainder of this paper.

The general plan of this crossing, Plate XIX, shows its location with reference to the principal streets and public buildings. It will be seen that a direct approach to the south end of the structure was prevented by the Court House and Jail buildings, which block Main Street at this point. It was desirable, therefore, to make the south approach a double one. One branch, carrying north-bound traffic, connects with Commerce Street, and the south-bound traffic is accommodated by the other branch, which connects with Houston Street. Sufficient property has been acquired to allow the street corners to be rounded off, so that the approaches may follow easy curves.

Curved retaining walls and railings are being constructed south of the south abutment, where the viaduct proper ends. It is expected that the block now occupied by the Jail and other buildings, will be cleared by the City, and will be parked to form a pleasing approach to the crossing.

The general elevation was laid out to fit the conditions, keeping in mind the data obtained from the preliminary designs. It will be seen by referring to this general elevation, Plate XX, that a symmetrical structure was not possible, owing to the contour of the ground.

Structural Features.—Before going into the matters of loading, unit stresses, and methods of design, it may be well to give a descrip-

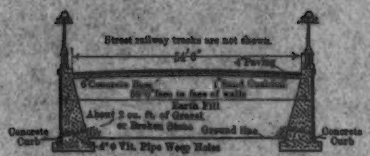
tion of some of the features of the Main Street Viaduct, in order to show the character of the construction, and to make clearer some of the problems involved in the design.

As shown by the plans, the deck consists of slabs carried on longitudinal stringers, which connect to floor-beams. These floor-beams are in turn carried by the four main longitudinal girders of the girder spans, or by the spandrel posts, which rest on the four ribs of the arch spans. The sidewalks are carried on cantilever extensions of the floor-beams outside of the outer girders or ribs. It will be noticed that, for the girder spans, such stringers as would come close to the girders are omitted, and their place is supplied by the cantilevered top flanges of the main girders. Those cantilevered top flanges also provide the necessary compressive area for the girders.

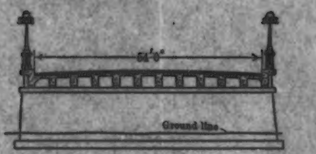
Two ribs or main girders would have been more desirable in some respects than four, as this arrangement would have done away with the continuous girder action of the floor-beams, and made the reactions on the girders and ribs statically determinate. The heavy concentrated loading and the width of the roadway, however, would have made the floor system extremely heavy. Therefore the four-rib arrangement was adopted for this crossing. At West Seventh Street, three ribs were used, and two lines of main girders for the other two structures.

The same length of floor system panels was adopted for all the structures. This made the design of the stringers practically uniform throughout. The length used was 12 ft. 6 in. from center to center of floor-beams, which gave stringers of reasonable size and brought the loads on the girders and arch ribs at reasonably short intervals. The floor-beams are of uniform thickness throughout, which thickness, 17 in., is the same as that of the spandrel posts. The posts are purposely made thinnest in the direction of the axis of the structure, so that the bending caused by the changes in the length of the deck due to temperature, will not over-stress them.

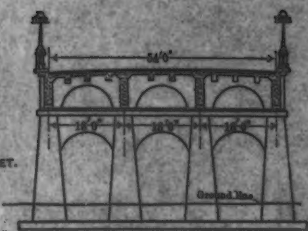
Expansion joints are provided in the girder spans at each pier, and in the deck of the arch spans, at the crown, and at each pier. The main girders slide on top of the small piers, and the stringers slide in recesses or pockets in the floor-beams. All expansion joints are packed with tar-paper and asphalt to form a closed but compressible joint, and all main girder joints are masked by pilasters. Planed steel, bed-



TYPICAL CROSS-SECTION THROUGH RETAINING WALLS.

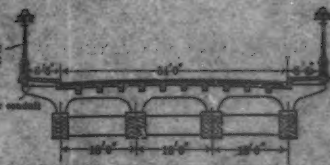


CROSS-SECTION THROUGH GIRDERS AT N. THIRD STREET.

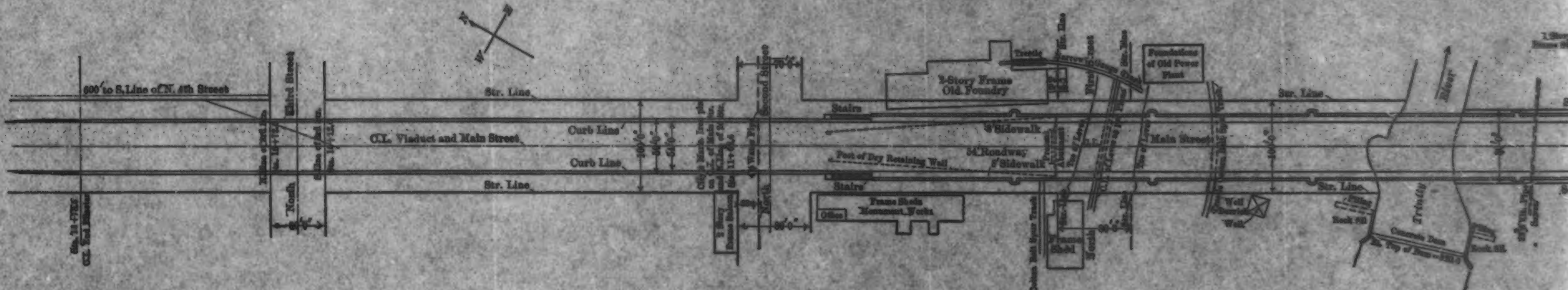


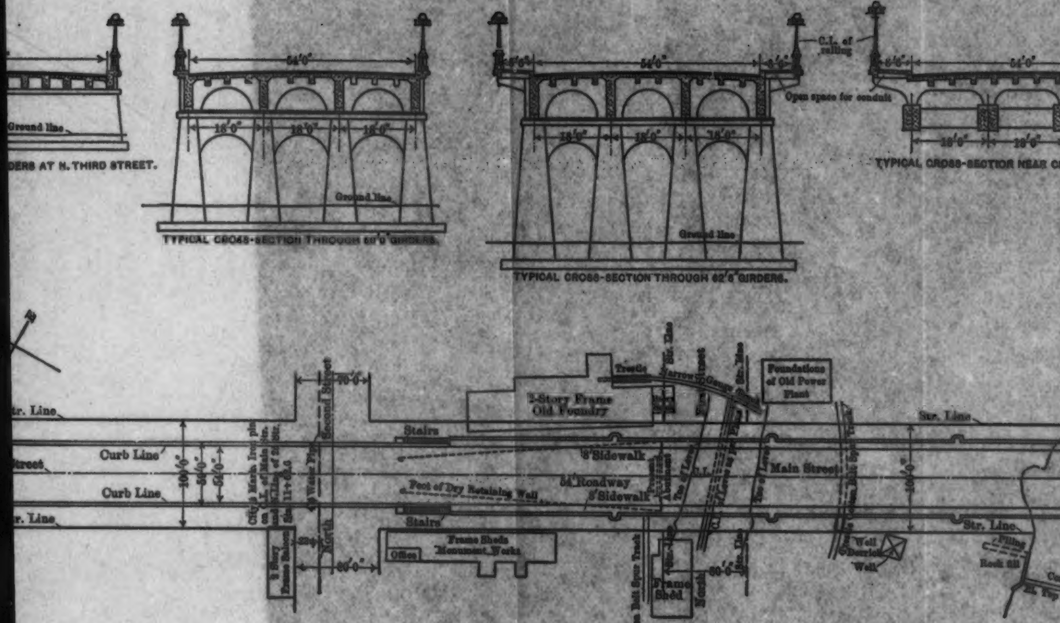
TYPICAL CROSS-SECTION THROUGH 50'0" GIRDERS.

TYPICAL CROSS-SECTION THROUGH 62'6" GIRDERS.



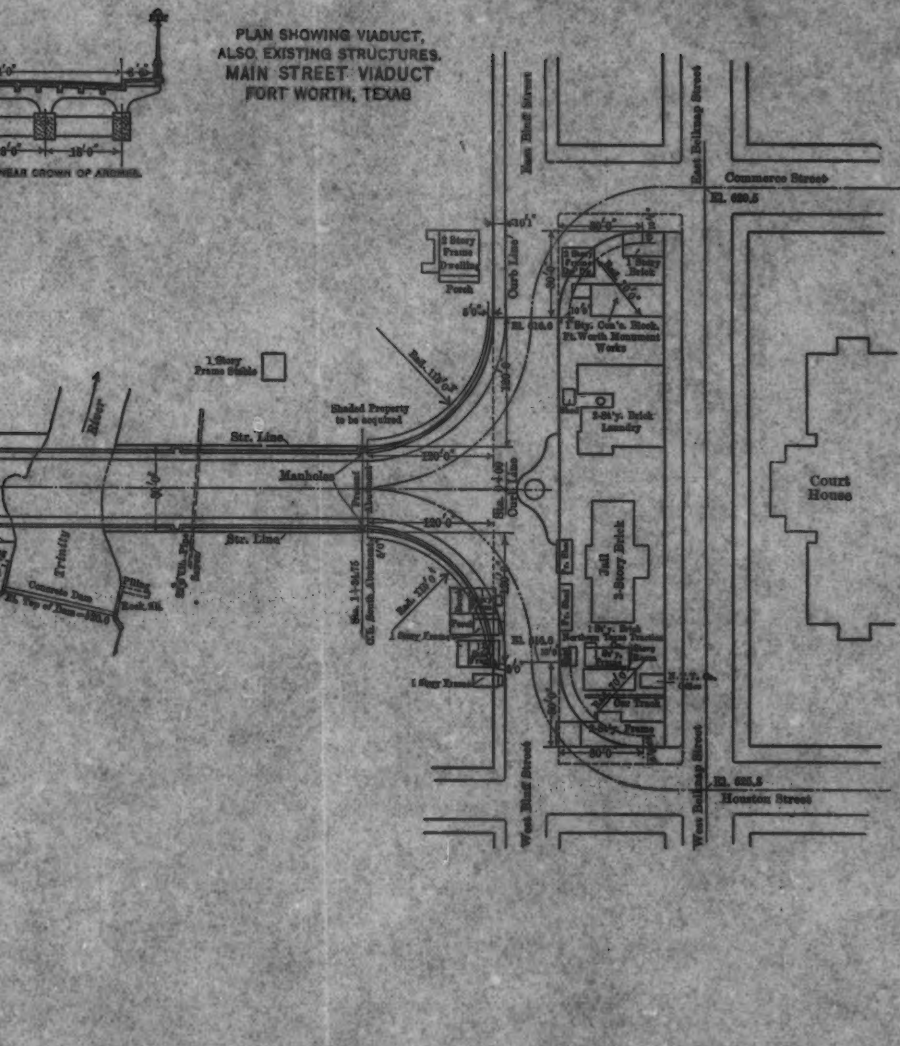
TYPICAL CROSS-SECTION NEAR CROWN OF ARCHES.





54'0"
Ground line
DERS AT N. THIRD STREET.
18'0"
Ground line
TYPICAL CROSS-SECTION THROUGH 62'6" GIRDERS.
C.L. of railing
Open space for conduit
TYPICAL CROSS-SECTION NEAR CR
Str. Line
Curb Line
Street
100'0"
Second Street
North
Stairs
8' Sidewalk
54' Roadway
Foot of Dry Retaining Wall
Frame Sheds Monument Works
Office
2-Story Frame Old Foundry
Trestle
Foundations of Old Power Plant
Main Street
Frame Shed
Cotton Belt Spur Track
Well
Derrick
Piling
Rock fill

PLATE XIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1829.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



and bearing-plates are used under the expansion end of each girder and stringer. These plates have anchorage angles riveted to them, and are further anchored with bent anchor-bolts passing through holes in these angles. The sliding members, and those on which the sliding members rest, are also reinforced at these bearing points with loops of steel, to provide for the shearing stresses which result from the friction of the plates on each other. In order to reduce this friction as much as practicable, the bed- and bearing-plates are planed in the direction of movement, and the sliding surfaces in contact are heavily coated with tallow. U-shaped copper plates, filled with asphalt, are used in the roadway slabs to close the opening. Similar plates placed in the reversed position to those in the roadway slabs, and with the asphalt filling omitted, are used in the sidewalk slabs and curbs. These copper plates are used at each expansion joint in the structure.

The arch ribs are braced together at the foot of each spandrel post by transverse braces, which are of the same width as the spandrel posts and floor-beams, and as deep as the ribs will allow. These braces are placed in vertical planes.

The arch ribs are of uniform thickness, but vary in depth, decreasing regularly from the haunch toward the crown. This is contrary to the theoretical shape of a three-hinged rib, which should be deepest at about the quarter point of the span, and diminish in depth toward the haunch and crown. This departure from the theoretical shape was made for the sake of appearance only. As the hinges are concealed in the finished rib, it was considered that the ribs would look best if made to follow the lines of a hingeless rib.

Each hinge consists of two steel castings, having ball and socket joints. At the haunches one of these castings is built into the pier, and the other is bolted to the structural rib reinforcement, as are also the castings forming the crown hinge.

All the piers for the arch spans are of considerable width in order to resist overturning forces and to give the appearance of massiveness required by the long spans. They are hollowed out as much as is consistent with strength and economy of forms, in order to reduce their weight and to save material.

The small piers supporting the girder spans consist of four square shafts, battered on the four faces. These shafts rest on a common base and are connected at the top by a cap and diaphragm of T-section.

The retaining walls are of the semi-gravity type. The toe of the section is reinforced for bending stresses, and the reinforcement extends high enough up the back of the wall to relieve the concrete of undue tensile stresses. Expansion joints are provided at intervals of 37 ft. 6 in., and pilasters are used at these points to mask the joints. The walls have a small coping and are paneled to relieve the appearance of flatness.

The South Abutment is founded on rock, as is also Pier No. 1, which is on the south bank of the stream. Pier No. 2, on the north bank, is supported on a spread foundation, with a portion of the load carried on timber piles. All other piers and walls are carried on spread footings, resting on the strata previously referred to, at depths ranging from 4 to 18 ft. below the finished ground surface.

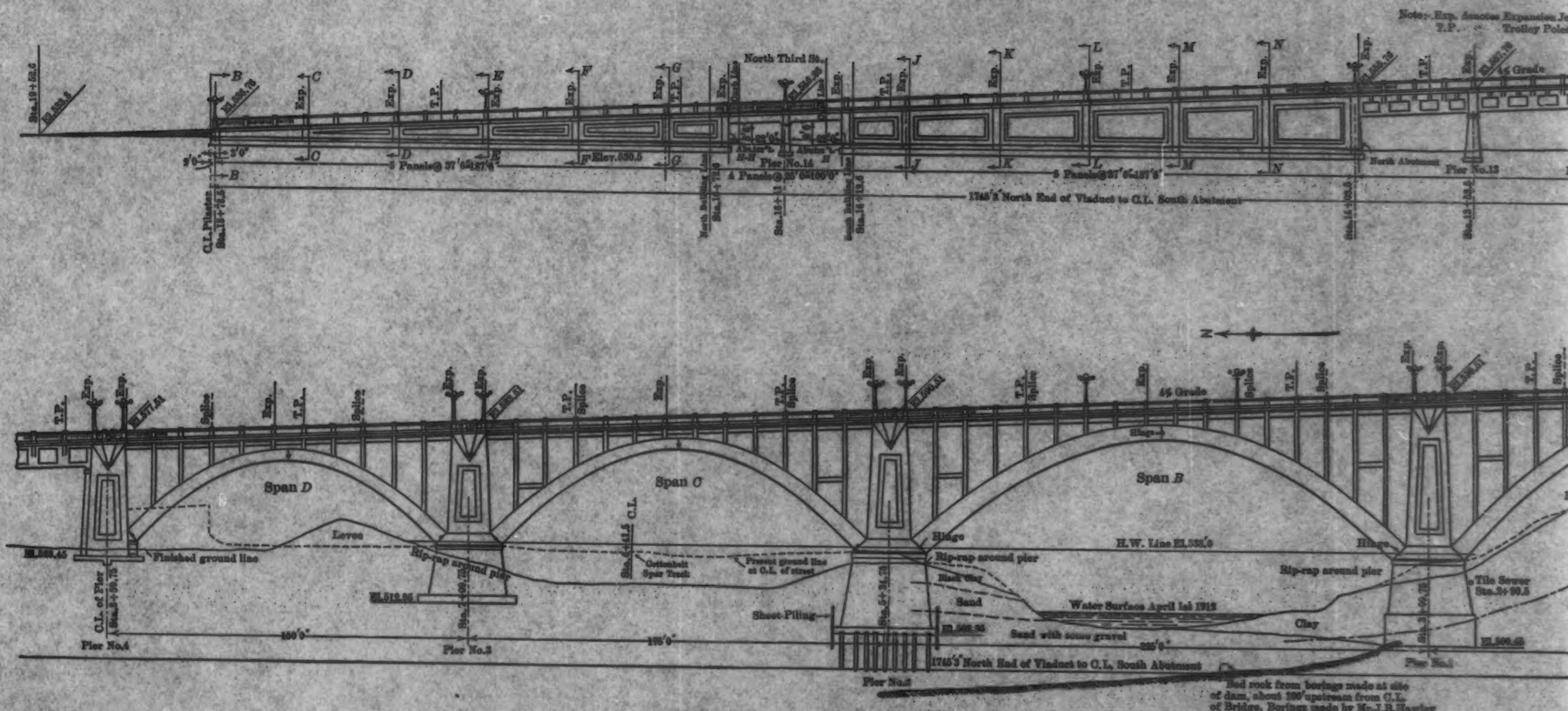
Railings, and the stairways leading down to the street level at North Second Street, are of reinforced concrete, cast in place, except the railing panels and hand-rails, which were cast in moulds and erected.

The stairways, with the exception of the steps leading to the lower landing, are supported entirely from the piers and girders by structural steel cantilever beams encased in concrete. This method was adopted, instead of supporting partly on the piers and girders and partly on independent posts and foundations, in order to avoid the danger of damage due to unequal settlement. The lower steps were built on independent foundations, with a sliding joint between the steps and the remainder of the stairway, to take care of unequal settlement.

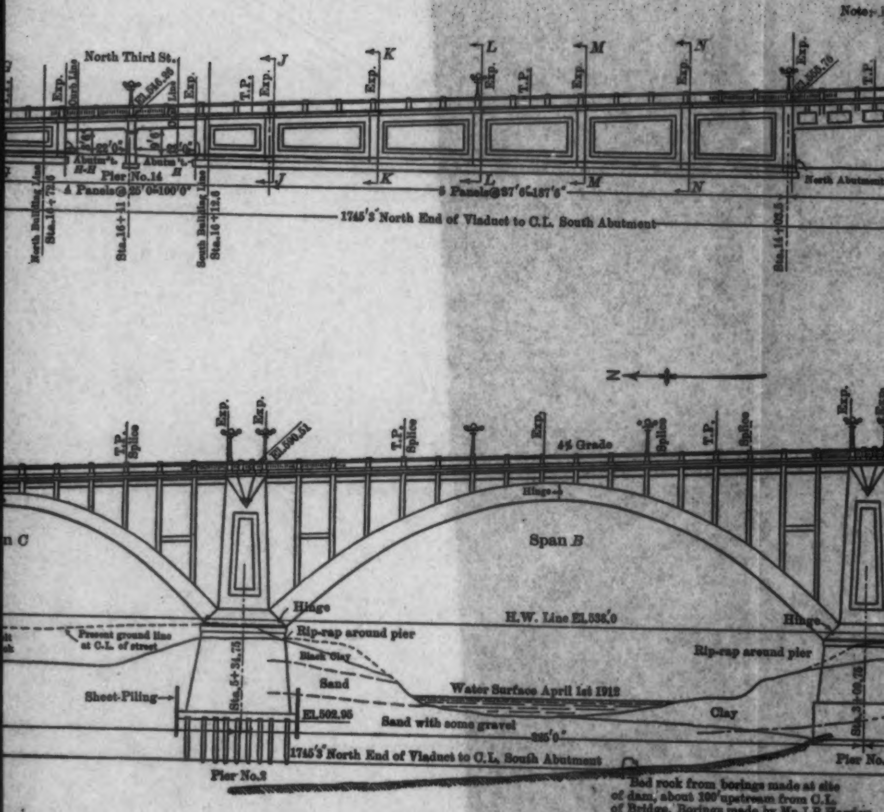
Provision is made for carrying electric wires and small pipes under the sidewalks, with ducts leading into the railing posts and up to the lighting fixtures.

In order to reduce the dead weight of the deck as much as possible, the usual ballasted track construction was dispensed with. Anchor-bolts were set into the slab over the stringers, and the rails were secured to these bolts with cast-steel clips. Steel bearing-plates were provided under the rails at each pair of bolts, which occurred at intervals of 27 in. The rails were lined and brought to proper level with eccentric washers and shim plates. A fiber pad, $\frac{1}{2}$ in. thick, was placed between the rail and the bearing-plate, as a shock absorber. This method of securing the rails is indicated on the plans.

The reinforcement of the arch ribs and braces consists of medium-steel structural shapes, supplemented by reinforcing bars. Structural

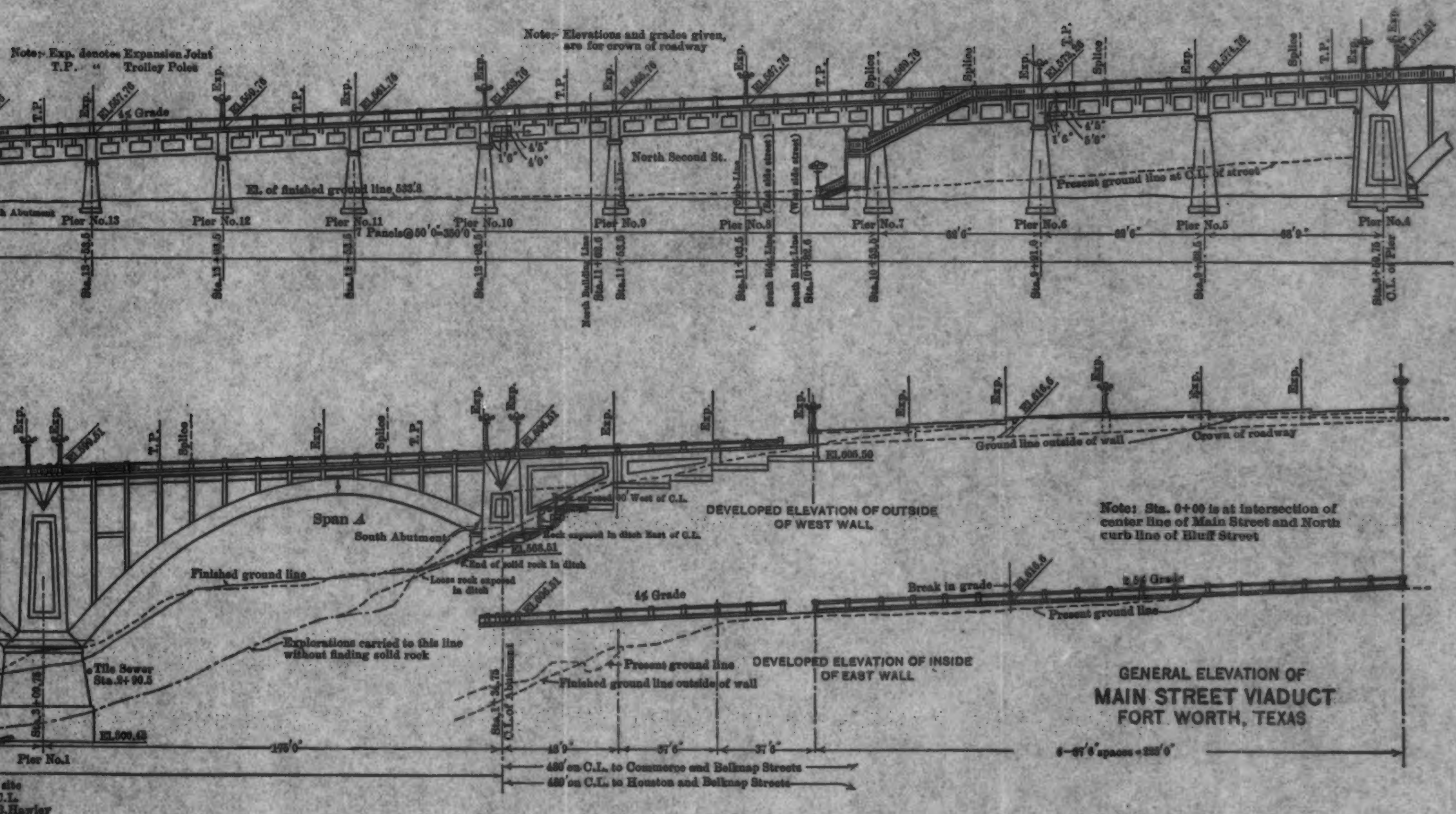


T.P. Trolley Poles
North Third St.
5 Panels@37'6"=187'6"
4 Panels@25'0"=100'0"
Elev.530.5
Pier No.14
North Abutment
Pier No.13
1745'3" North End of Viaduct to C.L. South Abutment
C.L. Pilaster
4% Grade
Span D
Span C
Span B
Levee
Finished ground line
Rip-rap around pier
Cottonbelt Spur Track
Present ground line at C.L. of street
H.W. Line El.533.0
Hinge
Black Clay
Sand
Water Surface April 1st 1913
Clay
Sand with some gravel
Sheet-Piling
Tile Sewer
C.L. of Pier
Pier No.4
Pier No.3
Pier No.2
Pier No.1
150'0"
175'0"
Bed rock from borings made at site of dam, about 100' upstream from C.L. of Bridge. Borings made by Mr.J.B.Hawley



Note:
North Third St.
Exp.
Curb Line
El.616.93
T.P.
Abutm't.
H-H
Abutm't.
H
Pier No.14
4 Panels@25'0"=100'0"
5 Panels@37'6"=187'6"
North Building Line
Sta.16+72.6
Sta.16+11
South Building Line
Sta.16+12.6
1745'3" North End of Viaduct to C.L. South Abutment
Sta.14+02.5
North Abutment
N
T.P. Splice
El.590.51
4% Grade
Hinge
Span B
Span C
H.W. Line El.538.0
Present ground line at C.L. of street
Rip-rap around pier
Black Clay
Sand
Sheet-Piling
Sta.5+34.75
Water Surface April 1st 1912
El.502.95
Sand with some gravel
Clay
Sta.3+09.75
325'0"
1745'3" North End of Viaduct to C.L. South Abutment
Pier No.2
Pier No.
Bed rock from borings made at site of dam, about 200' upstream from C.L. of Bridge. Borings made by Mr.J.B.Hawley

PLATE XX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1329.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



GENERAL ELEVATION OF
MAIN STREET VIADUCT
FORT WORTH, TEXAS

shapes are also used for the main supports of the stairways. All other reinforcement consists of square twisted bars, of high elastic limit. These were substituted for the round, mechanical-bond bars called for on the plans, and were made equivalent in cross-section to the round bars which they replaced. All bars, except some of the large sizes, were rolled from the heads of rails, and were hot-twisted. Cold-twisted bars, from medium-steel billet stock, were used for the large sizes, on account of the difficulty of getting rail heads of sufficient volume for this purpose. Tests of the two classes of material showed that they were practically equal in quality.

As shown on the plans, the rib reinforcement consists of curved lattice girders, to the ends of which are bolted the cast-steel hinges. The chords of these girders are made up of four angles arranged in the shape of a cross. All web members are made up of two angles. Three of these steel girders are used in each inner, and two in each outer, rib of each span. The individual steel girders are laced together in the plane of the top and bottom chords, and are also connected by transverse frames, in vertical planes, under each post. The transverse braces connecting the four ribs are reinforced by lattice girders which connect to the rib reinforcement. No reaming or painting is called for. The hinge castings are bolted to the steel ribs, and all other connections, both shop and field, are riveted.

At each upper panel point of the steel ribs, clip angles were provided, to which were secured the transverse timber beams, from which the rib forms were supported. The method of supporting these forms is indicated on the plan showing the arch rib reinforcement.

A system of lateral rods, for use during erection, was provided in the plane of each chord of the ribs. These rods were used to line up the steelwork accurately, and to hold it in line while the ribs were being concreted. The rods were not removed until after the completion of the work. The transverse braces, mentioned previously, are also reinforced in each face with continuous bars, the function of which is to resist bending in the braces due to transverse forces.

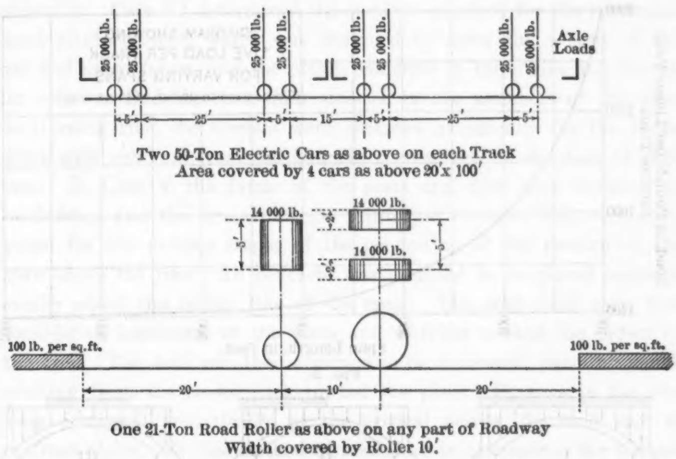
In addition to the steel reinforcement required to resist the calculated stresses in the members, a certain quantity of steel is used to prevent the formation of shrinkage, settlement, and temperature cracks. This steel is used in the exterior faces of thin walls, in a longitudinal direction in the slabs, and to reinforce the corners where

two walls join. All braces, ribs, and posts are wrapped with heavy wire closely spaced, to which the longitudinal steel is secured.

The concrete used in the construction of the viaducts is of three classes: No. 1 concrete, used for floor systems, girders, arch ribs, braces, spandrel posts, railings, stairs, and rib seats of large piers, composed of 1 part Portland cement, 2 parts sand, and 4 parts broken stone; No. 2 concrete, used for retaining walls and abutments, girder piers, and large arch piers (except rib seats), composed of 1 part Portland cement, $2\frac{1}{2}$ parts sand, and 5 parts broken stone; No. 3 concrete, used only for paving foundations, composed of 1 part Portland cement, 3 parts sand, and 6 parts broken stone. A local sand of good quality was used, and the broken stone was a hard limestone of the same quality as that from the quarries near Jacksboro, Tex. The maximum sizes of stone for the various classes of concrete are as follows: for No. 1 concrete, used for railing, $\frac{3}{4}$-in., all other No. 1 concrete, 1-in.; for No. 2 and No. 3 concrete, 2-in. The stone was graded below these maximum sizes, and the dust was screened out. A very dense concrete was obtained.

Loading.—As all the viaducts are in or near the city, and are likely to carry the same class of traffic eventually, the same live loading was used throughout. The loading data are shown by Figs. 4 to 7, inclusive. The 50-ton electric cars and the 21-ton road roller shown thereon, were used for the members of the floor system, together with a uniform load of 100 lb. per sq. ft. on the sidewalks and remaining roadway surface. For maximum stresses in the floor-beams, the sidewalks were assumed to be unloaded. The arch ribs and the main longitudinal girders were designed for the foregoing car loading, together with 100 lb. per sq. ft. on the sidewalks and the remainder of the roadway. The road roller was also considered in connection with the car loading, with the foregoing uniform live load on the sidewalks and the remainder of the roadway. A load of 200 lb. per lin. ft. of each sidewalk was also used, in order to provide for the weight of pipes and conduits that might be carried in the spaces provided for them under the sidewalks. Pier footings were designed for a uniform load per linear foot of each car track, as given on Fig. 5, together with a uniform load of 100 lb. per sq. ft. on the sidewalks and the remaining roadway surface.

Impact was allowed for according to the following formula: $I = \frac{100\,S}{L + 300}$, where I is the impact to be added to the live-load stress, S is the calculated maximum live-load stress, and L is the loaded length, in feet, that produces the maximum stress in the member.



The car and roller given above, together with 100 lb. per sq. ft. on sidewalks and remainder of roadway, to be used in design of floor system, main longitudinal girders, and arch ribs. For Piers and footings, use 100 lb. per sq. ft. of sidewalks and Roadway and uniform track loading as per diagram, except Case II in design of arch Piers, where wheel loads are to be used to give maximum thrusts.

IMPACT

On sidewalks	none
Road Roller	25%
Other roadway loads and track loads	$I = S\,\frac{100}{L+300}$

where I = Impact to be added to the Live Load Stress.
S = Calculated maximum Live Load Stress.
L = Loaded Length, in feet, which produces maximum stress in the member.
No impact to be used in design of Piers and footings.

FIG. 4.

This impact allowance is one-third of that required by the original formula, submitted by the American Railway Engineering and Maintenance of Way Association, for impact on steel railway bridges, and was arbitrarily assumed. This formula was used only for car and

uniform roadway loads. Stresses produced by the road roller, which is a slowly moving load, were increased by 25% in all cases. No addition was made for impact to the stresses produced by the uniform sidewalk loading, as it was considered that when this sidewalk load was a maximum, that is, due to a closely packed crowd of people, it neces-

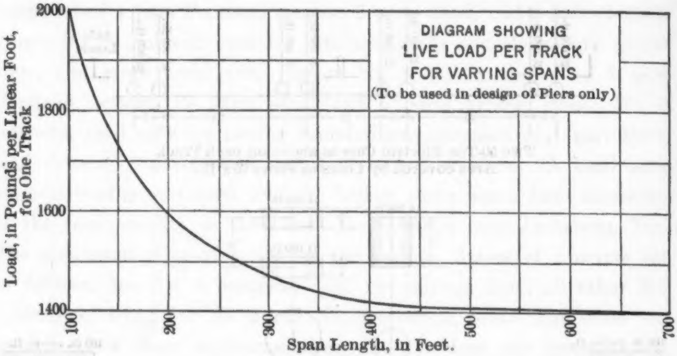
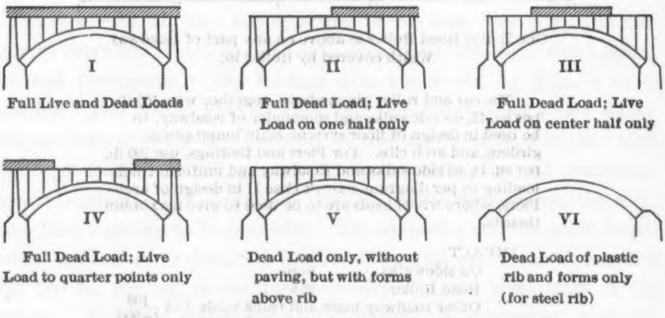


FIG. 5.



LOADING TO BE USED IN DESIGN OF ARCH RIBS

Note: In all cases where Live Load is called for, Wheel Concentrations shall be used and Allowance included for Impact

FIG. 6.

sarily would be nearly quiescent. The live loads for the piers were not increased for impact, as it was believed that the impact would be absorbed before reaching the footings.

The various conditions of loading for which the arch ribs were designed, are indicated on Fig. 6. Cases I to IV require no explana-

tion, except to say that they served to produce maximum stresses in the ribs of all spans except those of the unsymmetrical arch, Span *A*. On account of this lack of symmetry, these cases were elaborated on, in order to be sure that the maximum stresses had been found. Cases V and VI are construction conditions, and were elaborated on considerably. Case VI determined the sections required for the structural steel reinforcement, which was designed to carry the weight of the rib and brace forms, and the plastic concrete in these ribs and braces. In order to find the maximum stresses in the members of the steel reinforcing ribs, the stresses were obtained graphically for the forms filled with concrete from the haunch, successively to the foot of each post. In Case V the forms of the posts and deck were assumed to be finished and the stresses in the reinforced concrete ribs were computed for the various stages of the concreting of the portion of the span above the ribs. All concrete was assumed to be placed symmetrically about the center line of the span. The post loads were first considered, beginning at the piers and working toward the crown of the arch. The deck was then assumed to be concreted, panel by panel, working from the crown back toward the piers. Stresses in the ribs were computed analytically, at the critical points, for each step as outlined above. By this method of procedure in concreting, the stresses in the ribs were kept within the limits set for working conditions in the finished structure. The arch piers were designed for the conditions indicated on Fig. 7, which needs no explanation. The piers for the girder spans were designed only for full vertical load.

In addition to the above mentioned vertical loads, the structures were designed to resist transverse forces due to wind and water pressure. The arch ribs and their braces, in particular, were designed with great care to resist these transverse forces. In general, in computing these transverse forces, the water line was assumed to be at the top of the levee. Below this line the pressure due to the current was taken at 200 lb. per sq. ft. of vertical projection of each arch rib and spandrel post. Above this line the wind pressure was assumed at 30 lb. per sq. ft. of vertical projection of the structure, up to a line 10 ft. above the top of the roadway. The force assumed for the current corresponds, according to the formula used, to a velocity of about 8 or 9 miles per hour, which is probably not far from the actual velocity at flood stage.

At the West Seventh Street crossing, where the rise of the arch is small, and the high-water line comes well up on the arch rib, and where considerable drift is carried by the stream, more severe conditions

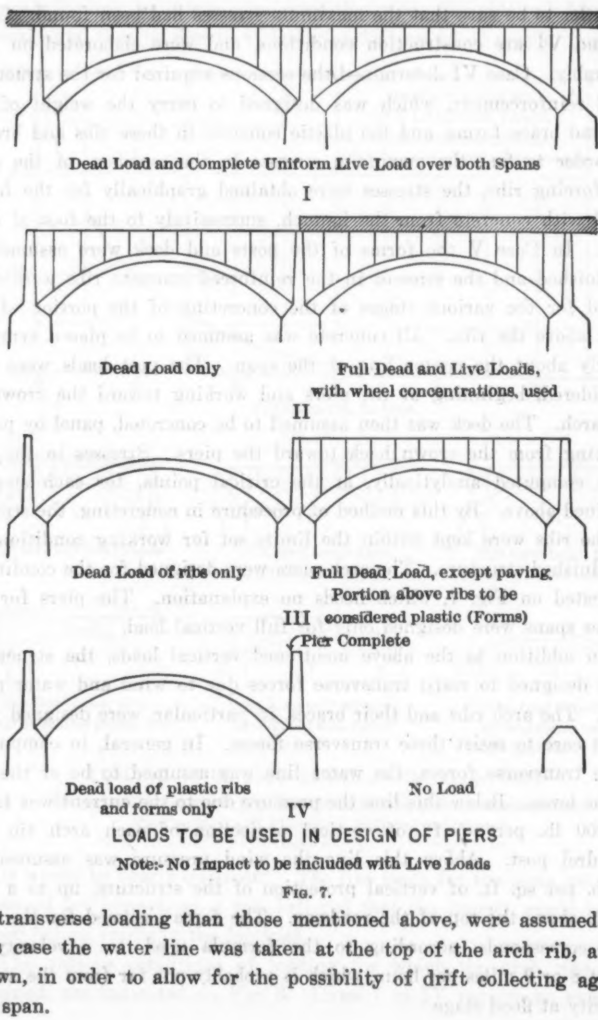


Fig. 7.

of transverse loading than those mentioned above, were assumed. In this case the water line was taken at the top of the arch rib, at the crown, in order to allow for the possibility of drift collecting against the span.

Specifications.—With certain modifications, in regard to unit stresses, "Watson's Specifications for Reinforced Concrete Bridges" were used.

The following is a list of the unit stresses used for the various parts of the structure:

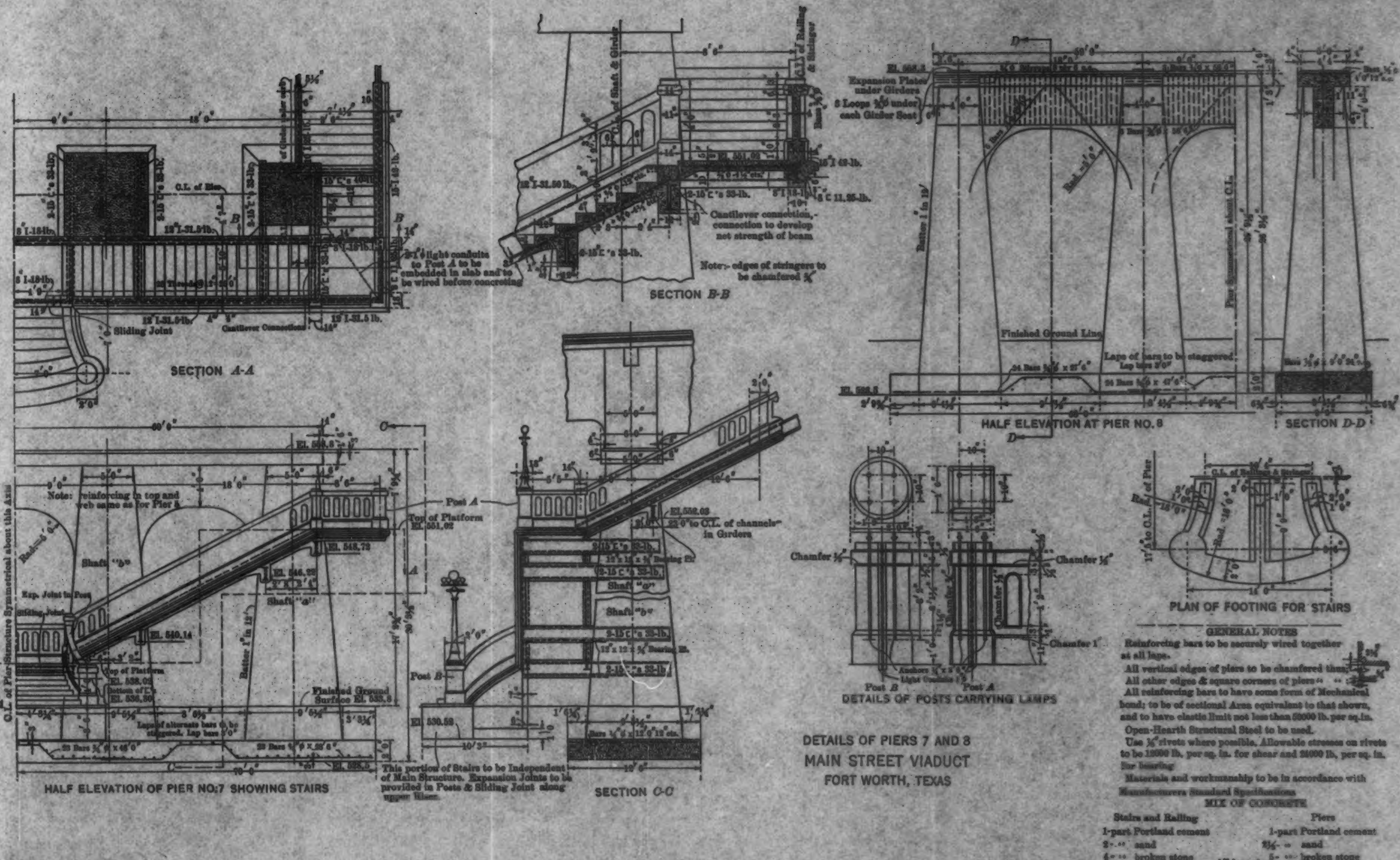
	Pounds per square inch.
Tension in steel bars used for flange reinforcement	20 000
Tension in steel bars used for web reinforcement	15 000
Compression in concrete in cross-bending, as in beams	750
Compression in concrete, combined direct and bending, as in arch ribs and posts	500
Compression in concrete of arch ribs for stresses due to combined vertical and transverse loads	650
Bond between concrete and steel, for deformed bars	120
Shear in concrete	60
Or, a maximum on the section, including shear taken by steel, of	150
Bearing of hinge castings on concrete specially reinforced	800
Compression in structural rib reinforcement due to weight of forms and plastic concrete in arch ribs and braces, 16 000–70 $\frac{l}{r}$, or a maximum of	12 500
Additional compression in structural rib reinforcement, under working conditions, and compression in steel of other compression members = 15 times the corresponding compressive stress in concrete. Possible maximum total compression in structural steel rib reinforcement	20 000
Tension on net section of structural steel reinforcement	16 000
Bearing, tension and compression in cast-steel hinges	20 000
(Bearing area of the hinges taken as the projected area of the hemispherical joint.)	
Bearing on rivets	24 000
Shear in cast steel	10 000
Shear on rivets	12 000
	Pounds per square foot.
Bearing on soil at depth of 6 ft. below finished surface	5 000
For each 1 ft. increase in depth, more than 6 ft., add	100
	Pounds.
Load on each timber pile	20 000

Methods of Design.—There is nothing unusual in the design of the structures, with the exception of the methods of designing the arch ribs and braces. Briefly, sidewalk slabs and main girders were designed as simple beams; and continuous members, such as floor-beams, stringers, and roadway slabs, were designed for 0.8 of the maximum positive moments, considered as simple beams. In all cases, where members were continuous, steel was provided over the supports to take the negative moments at these points. The quantity of this steel was two-thirds of that required to take the maximum positive moments in the members. The shears and reactions for continuous members, such as floor-beams and stringers, were increased by from 10 to 25% (depending on the number of spans) more than what these functions would have been for simple spans.

In order to make clear the methods used in the design of the arch ribs, braces, and hinges, a detailed account of this part of the work will be given.

It will be seen that the structural steel arch rib reinforcement takes stress in two independent steps: First, as the ribs are filled with concrete, the members take certain initial stresses, which, as the concrete sets, remain in the steel. This initial stress in the steel was not allowed to exceed a maximum of 12 500 lb. per sq. in. in compression, as given in the list of unit stresses. Second, after the ribs are finished and the construction of the posts and deck is under way, and later, when the structure is carrying live load, stresses are set up in the ribs, which are carried jointly by the steel and concrete in the usual manner. The assumed ratio of the moduli of elasticity of steel and concrete was taken as 15. Therefore, the maximum possible compressive stress in the structural steel reinforcement of the ribs would be 12 500 lb. plus 15 times 500 lb., or 20 000 lb. per sq. in. This is based on the assumption that the ribs and braces are completely filled with concrete in so short a time that it does not begin to set before all the load is in place, and also that the stress in the concrete at the center of gravity of the steel chord is 500 lb. per sq. in. As a matter of fact, the ribs were concreted in sections, and the braces were not concreted until the concrete in the ribs had set for several days. Consequently, the initial compression in the structural reinforcement was much less than that mentioned above. The center of gravity of the steel chords is some distance in from the face of the concrete, so that the concrete

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1829.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



stress at this point, and therefore the stress in the steel, is lower than that assumed. The actual maximum compression in the structural reinforcement is about 16 000 lb. per sq. in.

The lines of pressure for the ribs were worked out graphically for the various positions of the live load, combined with dead load, and for the various stages in the construction of the deck. In order to allow for the weight of the ribs themselves, their dimensions were first assumed, and later were revised where necessary. The neutral axes of the ribs were drawn midway between the extreme pressure lines. This made the neutral axis of the rib a constructive curve, approximating a parabola with its vertex at the crown of the arch. The axis as drawn followed closely the pressure line for full dead and live load. The inner and outer ribs were made of the same vertical dimensions, but of different thickness.

The next step was to find the stresses in the reinforced concrete rib for the various stages in the construction of the deck, and for the dead and live load, under working conditions in the finished structure. The quantity of steel required at each point to keep the extreme fiber stress in the concrete down to 500 lb. per sq. in. for the worst condition, was then computed. The required area of steel in either face of the rib at any point was not allowed to exceed 2.5% of the total cross-sectional area of the rib. Where, on first trial, the required quantity of steel exceeded the above maximum, a re-design was made, using a rib of larger cross-section.

As mentioned under the head of "Loading", the stresses in the structural steel ribs were found graphically for the various positions of load during the concreting of the ribs and braces, using the rib having the dimensions which had been determined as just described. The sections of the structural steel reinforcement were then determined. This structural steel reinforcement, though designed to carry the ribs and braces during their construction, also forms part of the reinforcement of the finished rib, as previously mentioned. Where the required area of steel in the finished rib exceeded that supplied by the structural reinforcement, the deficiency was made up by the addition of reinforcing bars. These bars do not run the full length of the ribs, but stop off where they are not needed. It will be noticed that these bars take little, if any, initial stress, such as is taken by the structural reinforcement. Additional bars are placed in the

sides of the ribs, running full length. These bars help to resist transverse bending stresses, and to prevent surface cracks from appearing.

The transverse forces of wind and water pressure, mentioned previously, were assumed to be resisted wholly by the ribs and braces acting as a series of portals, in the same manner as the columns and girders of an office building act to resist wind pressure. The deck was not given credit for any resisting power, because of the presence of the expansion joints over the haunch and crown of the span.

Points of inflection were assumed in the braces, midway between each pair of ribs, and in the ribs, midway between the braces. The ribs being hinged at the haunches and crown, were computed as being free to move at these points. Bars were provided at the haunches, however, anchoring the ribs to the piers, and at the crown, anchoring the two halves of the rib together. These bars were placed in the plane of the neutral axis of the rib, and tended to resist transverse, but not vertical movement, of the ribs. The stresses in the ribs and braces were computed at each section, for the moments, shears, and direct thrusts in the members. The maximum stresses at each point in the ribs, due to transverse forces, were combined with the maximum stresses due to the vertical loads, and the ribs were proportioned accordingly.

The design of the hinges naturally received considerable attention. At first, a hinge was studied which consisted of two castings, having semi-cylindrical bearings, and a pin. This type has a number of advantages, but was finally abandoned on account of the difficulty of setting the hinges on the piers with sufficient accuracy. Unless the axes of the pins are placed exactly at right angles to the vertical plane through the longitudinal axis of the rib, eccentric stresses will result in the rib, which may be too large for safety. It was finally decided to use hinges composed of two ribbed steel castings, having a hemispherical ball and socket joint, as shown on the plans. This joint was machined, and, by allowing movement in all directions, made great accuracy in setting the castings unnecessary. This joint also made it possible to adjust the structural rib reinforcement properly in vertical planes, with the temporary adjustable lateral rods provided for that purpose. This type of hinge reduces to a minimum the possibility of eccentric stresses in the ribs.

PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
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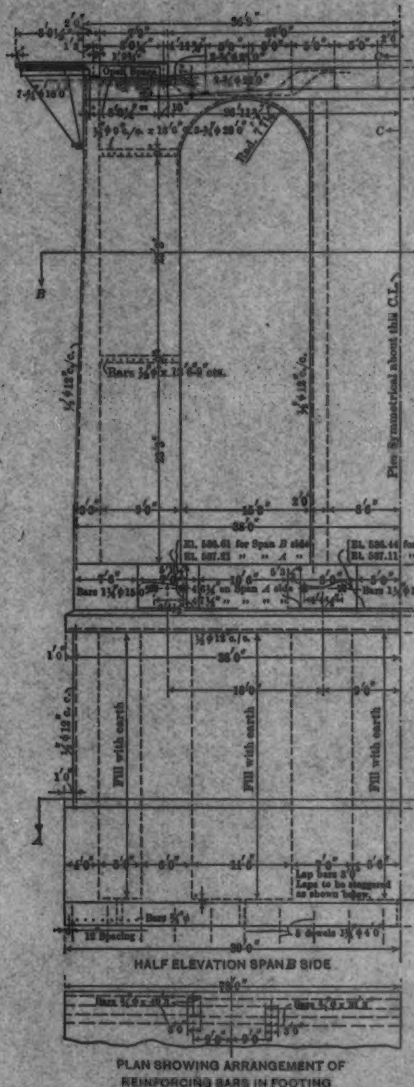
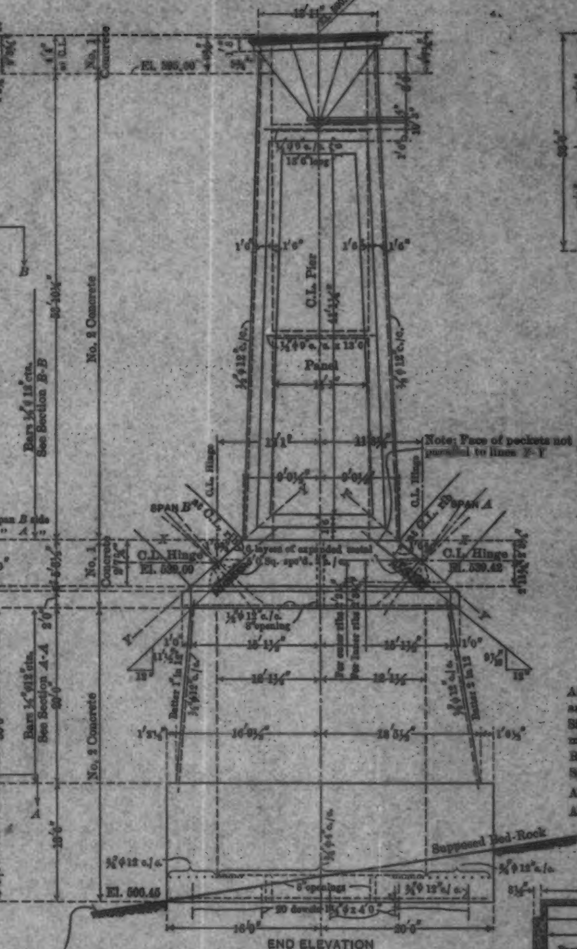
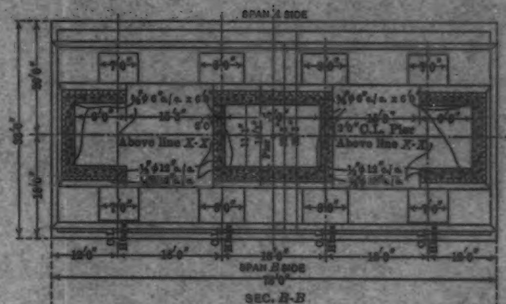
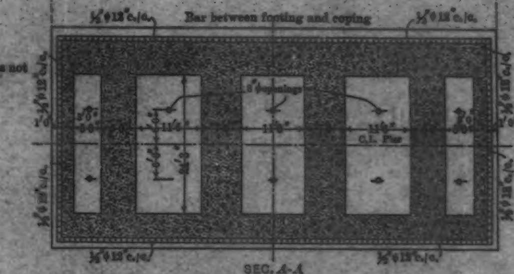
BOWER ON
REINFORCED CONCRETE VIADUCTS.





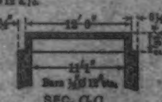
NOTE
Bed-rock to be benched out in horizontal steps and footing course of pier to be built to suit these steps as directed by the engineers.





GENERAL NOTES

All reinforcing bars to have some form of mechanical bond and to be of sectional area equivalent to that shown and to have elastic limit not less than 50000 lb. per sq. in.
Six layers of expanded metal 5′0″ square with 3″ mesh of No. 10 standard gauge metal placed back of each hinge.
Recessed portion of panels to be bush-hammer dressed.
Span A side of pier same as span B side unless otherwise shown.
All vertical edges of pier to be chamfered thus 2½″
All vertical edges & square corners of copings to be chamfered thus 1½″



Mix for No. 1 Concrete	Mix for No. 2 Concrete
1-Part Portland Cement	1-Part Portland Cement
2 Parts Sand	2½-Parts Sand
4-Parts Broken Stone	5-Parts Broken Stone
All by Volume	All by Volume

DETAILS OF PIER NO. 1
MAIN STREET VIADUCT
FORT WORTH, TEXAS

Bearings similar to these hinges had previously been used for some slow-moving multiple trucks, carrying extremely heavy loads. In this case, movement in all directions was necessary to take up the inequalities in the track. These bearings gave perfect satisfaction.

In proportioning the hinge castings, much the same method was followed as in the design of a column base. A section was taken through the diagonal rib of the hinge, at the junction of the rib with the body, or shaft, of the casting, and the tensile, compressive, and shearing stresses on the section were computed. A section was also taken through the center of the hinge, and the stresses on this section were computed, on the assumption that the reaction on the base of the casting produced moments at right angles to this section, instead of radially, as had previously been assumed. The resultant of the radial and direct compressive stresses at the circumference of the shaft of the casting, was also investigated.

The camber of the arch spans was computed for full dead and live load, with impact, combined with temperature and an allowance for shrinkage. The rise of the structural ribs was increased by the amount of the camber. The total calculated camber for the longest span was about 3½ in.

In the design of the main piers, considerable attention was given to the question of protection against scour. Pier No. 2 was given particular attention because of the fact that it is only a short distance below a power dam having a fall of about 15 ft. The bottom of the footing course was placed at about 5 ft. below the bed of the stream. Oak piles were provided, extending about 15 ft. below the bottom of the footing. The sheet-piling for the coffer-dam was driven about 8 ft. below the bottom of the footing, and was left in place. The banks of the stream at this point were also protected by a large quantity of rip-rap. These precautions, together with the fact that the material in the river bed at this point is a stiff clay, not likely to erode, were believed to afford ample protection against scour.

Several different designs and estimates were made for Pier No. 2 before a satisfactory solution was reached. The following were considered: First, a pier similar to Pier No. 1, founded on bed-rock; second, a pier having its base at about the level of the bed of the river, with the vertical loads carried on eight cylinder piers, two under each line of arch ribs; third, a pier resting on piles driven to rock;

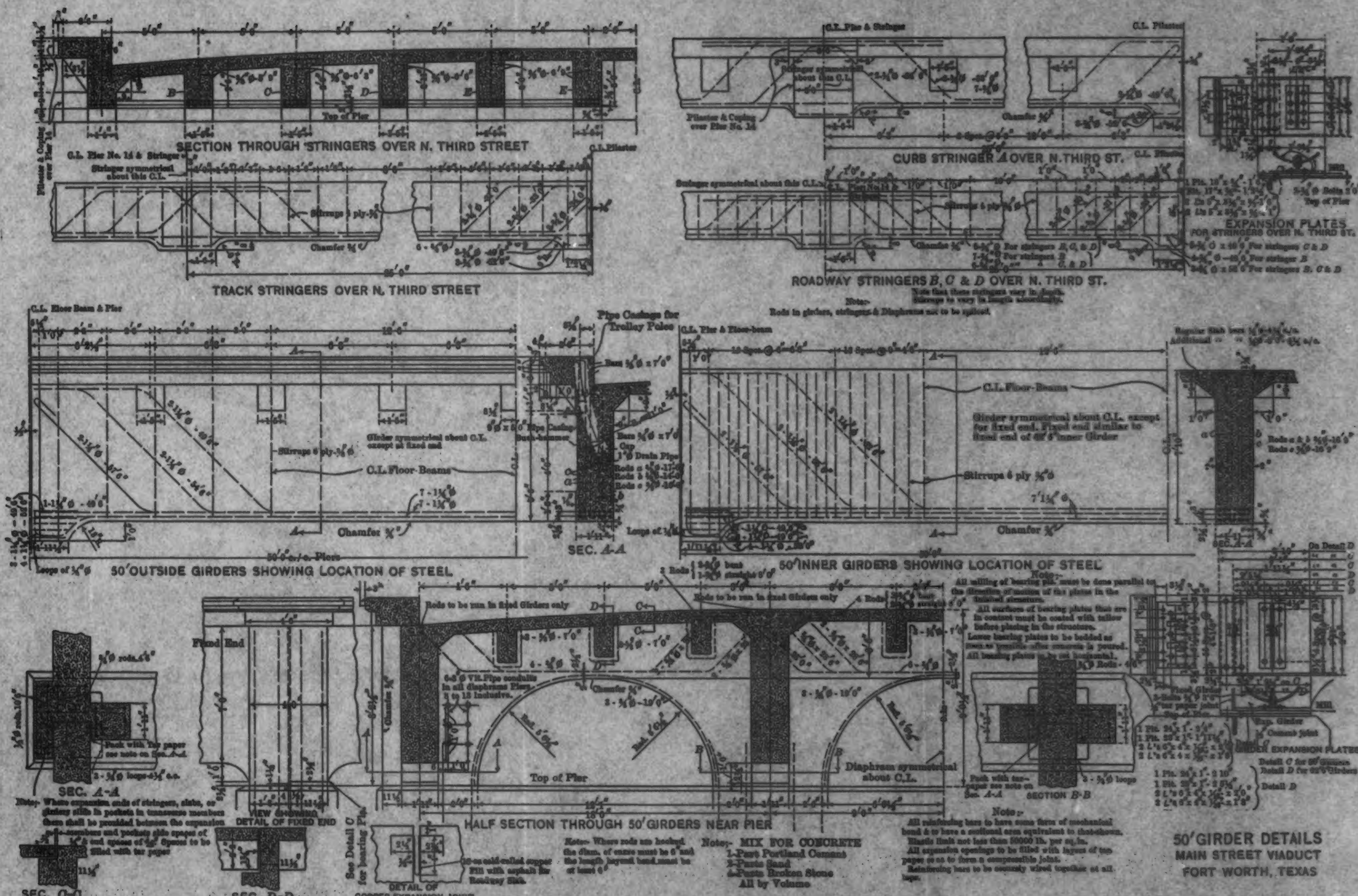
and fourth, the pier as built, having moderately spread footings, but with a portion of the load carried on piles which do not extend to rock.

The first scheme was abandoned because of the cost, which would have made that of the structure exceed the appropriation. The second scheme was considered inadvisable because of the difficulty of taking care of the unbalanced horizontal thrust. As the vertical load would have been carried down the cylinders to rock, the friction between the pier base and the material under it could not safely be counted on to resist the horizontal thrust. There then remained, to resist this thrust, only the pressure of the earth against the body of the pier and against the cylinders. Computations indicated that this resisting pressure would be insufficient, unless the body of the pier was extended to a considerably greater depth. This would have made the cost too great. The third scheme was rejected for practically the same reasons as those given above for the second scheme, and also because, after investigation, it was considered doubtful whether piles could be driven to rock through the mixture of sand and gravel overlying it. The fourth scheme proved to be the cheapest of those considered, and was believed to be the most feasible. Therefore it was adopted, with the provisions mentioned above to protect the pier from scour.

The small piers were designed for full vertical load only, using the unit pressures mentioned previously.

Rankine's theory of earth pressure was used in the design of the retaining walls. The weight of the earth fill was taken at 100 lb. per cu. ft. A superposed load of 100 lb. per sq. ft. was assumed, to take care of the roadway and sidewalk loads. For convenience in computing the pressures, the unit weight of the fill was considered to be increased by the ratio, $\frac{d'}{d}$, where d is the actual depth of fill, and d' is the actual depth plus the depth of the imaginary fill required to produce the same unit vertical pressure as the superposed load. In the foregoing case, $d' = d + 1$ ft. For the higher sections, this increase is negligible, but, for the lower ones, it is worth taking into account.

In concluding this portion of this paper, a few words in regard to the advantages of the three-hinged, ribbed arch may be in order.



50' GIRDER DETAILS
MAIN STREET VIADUCT
FORT WORTH, TEXAS

So much has been said on this subject, that little that is new can be brought out. However, to go over the ground again briefly, it is evident that the stresses in an arch of this type are statically determinate, and can be computed with as much accuracy as the loading assumptions and computations of the dead weight warrant. Temperature stresses and those due to settlement are eliminated, and the computations, though somewhat long, are simple. Practically, the only disadvantage is the cost of the hinges. In the Main Street Viaduct this item amounted to about 3½% of the total cost. This is more than it should have been, owing to the high price paid for the steel castings. Even at this price, when all the advantages of the three-hinged arch are considered, the money paid for hinges is well spent. The ribbed arch has two great advantages over the barrel, or solid arch. These are lightness, with consequent saving in cost, and the fact that water-proofing is not required. The latter, alone, is enough to recommend it to those who have had trouble with this feature of arch construction.

The increasing use of the three-hinged ribbed arch, both in the United States and in Europe, is evidence of its many advantages.

Construction.

Contractor's Plant and Equipment.—Work on the Main Street Viaduct started early in December, 1912, and was finished in March, 1914. The climate of Fort Worth is such that concreting can be carried on at all seasons of the year. The only interruptions were those due to rains, which were infrequent.

The general contractor's plant and equipment consisted of a central mixing plant, two distributing towers, with hoists, chutes, etc., an electric locomotive and cars for transporting concrete, derricks and clam-shell buckets for handling excavation and materials, a pair of driver leads suspended from the boom of a derrick, and the usual equipment of smaller tools and appliances. The sub-contractor for the structural steelwork had a steel stiff-leg derrick, a derrick car, two gin-poles, a two-bent timber gantry traveler, and the necessary hoisting engines and minor appliances. With the exception of one hoisting engine, all the general contractor's equipment was electrically driven, current being obtained from the local power com-

pany and transformed at the site. The sub-contractor's equipment was operated by steam.

While the pier excavations were under way, the mixing plant was being built near Pier No. 3 on the down-stream side of the structure. This consisted of an elevated hopper divided into two parts, one for rock and one for sand, a measuring box for sand and one for stone, and a mixer capable of handling a batch of a little more than 1 cu. yd. of concrete. Behind the mixing plant was a stiff-leg bull-wheel derrick for unloading materials and for supplying the hoppers of the mixing plant. Materials were brought to the site on cars, on a spur-track which crossed the line of the viaduct alongside of the mixing plant.

Concrete was discharged from the mixer into steel hopper cars drawn by an electric locomotive running on a narrow-gauge track. The concrete was carried in these cars to the two distributing towers, where it was hoisted and conveyed through the chutes to its destination. The larger of these towers was stationary, and was about 200 ft. high. This tower served the portion of the viaduct between Pier No. 4 and the South Abutment, except where concrete was discharged directly into the footings of Pier No. 3 from the mixer. The other and smaller tower was built so that it could be moved when required. It served the portion of the structure between Pier No. 4 and the North Abutment. For the retaining walls, concrete was carried in the hopper cars to the north end of the walls, where it was switched back on a trestle midway between the walls, and having the track parallel to and about 6 ft. above the tops of the walls. From this trestle the concrete was passed through chutes into the walls.

Construction Methods.—Steel forms were used where there was sufficient duplication to make them economical. They were especially advantageous on the girders and floor system, which had been laid out with the view of making the work as simple, and giving as much repetition, as possible. For instance, all floor-beams were made alike, so far as the dimensions of the concrete were concerned; all panel lengths, and consequently all stringers of each kind, were alike. The wall forms were made up of squares and rectangles of such size that they could be handled readily by two or three men. These forms were stiffened by angles, and were bolted together by bolts passing through the flange angles. They were braced by timber verticals and

PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
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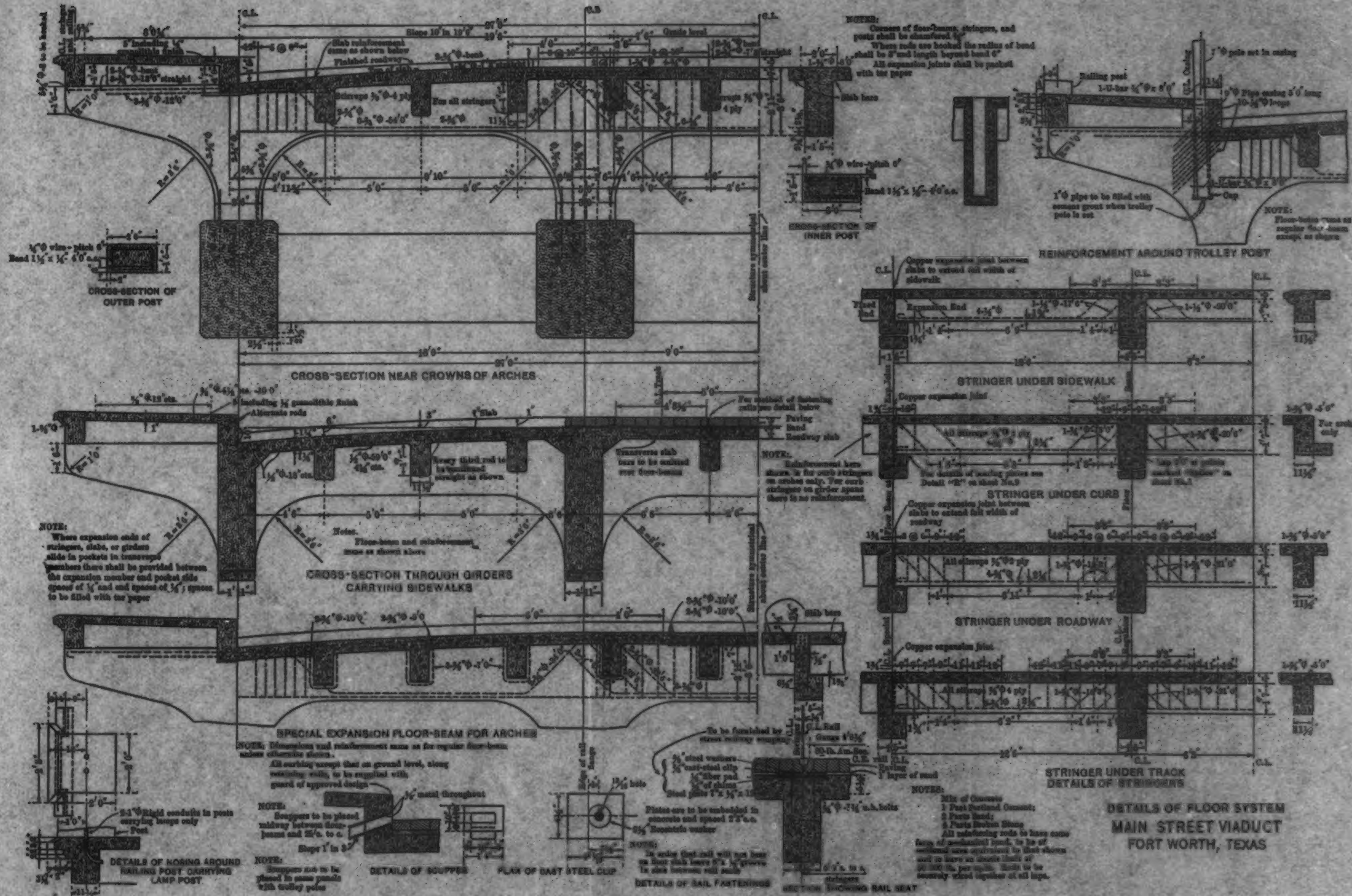
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



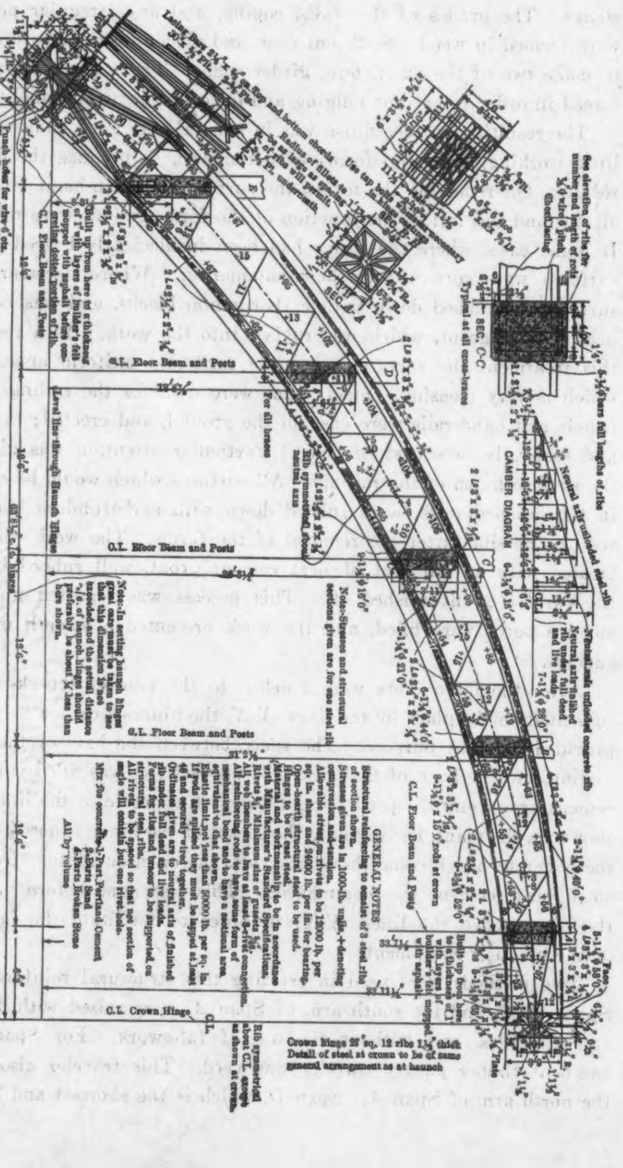
DETAILS OF FLOOR SYSTEM
MAIN STREET VIADUCT
FORT WORTH, TEXAS

shores. The grades of the walls, coping, and any irregular portions were formed in wood. Sufficient deck and girder forms were provided to make two of the 62 ft. 6-in. girder spans. These forms were reinforced in order to prevent bulging and deflection between panel points.

The resulting surface finish was in general very good, and required little working, after the forms were removed, to produce the desired results. On removing the forms, the surface was bush-hammered, and all fins and lips left at the junction of the form sections were removed. In some cases where the forms had been insufficiently braced untrue surfaces were corrected by bush-hammering. Where necessary, the surface was rubbed down with carborundum blocks, and washed with neat cement grout, which was rubbed into the work. As a result of this treatment the surfaces present a neat and uniform appearance, which is very pleasing. Steel forms were used for the railings. The panels and hand-rails were cast on the ground, and erected; the posts and foot-rails were cast in place. Particular attention was given to the surface finish of the railings. All surfaces which would be exposed in the finished work, were rubbed down with carborundum blocks as soon as possible after the removal of the forms. The work was then treated with a thin coat of neat cement grout, well rubbed in, and the surplus grout washed off. This process was repeated until the surface pores were filled, and the work presented a smooth uniform surface.

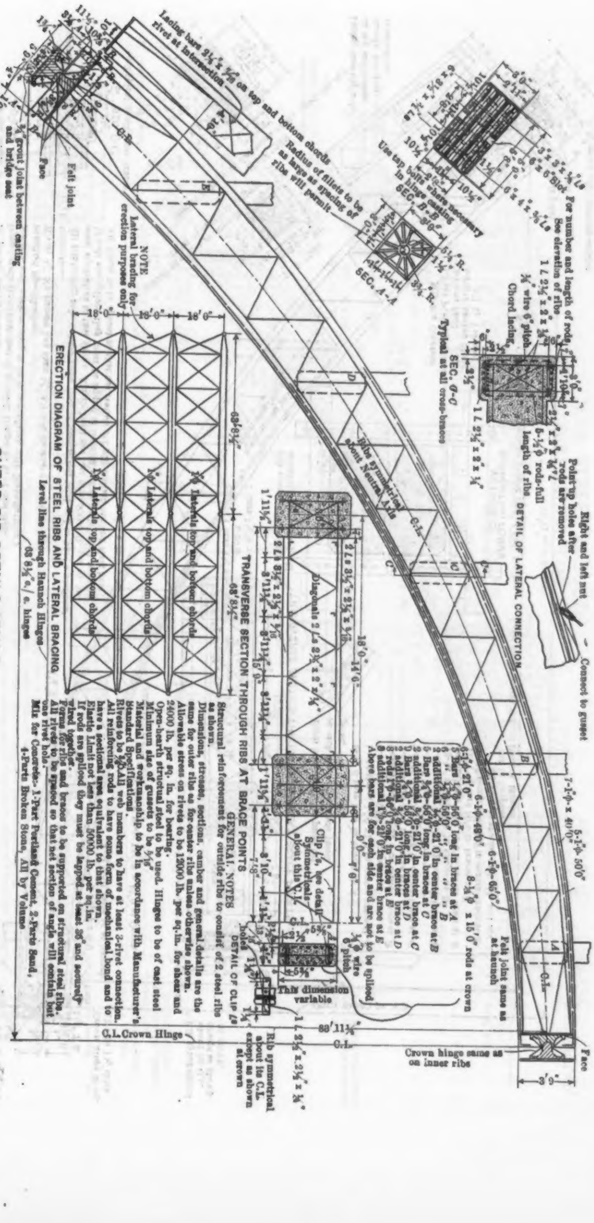
After the main piers were finished to the tops of the skewbacks, indicated on the plans by the lines, *X-X*, the hinges were set in pockets provided for that purpose. The space between the base of the hinge casting and the back of the pocket, of about $\frac{3}{4}$ in., was filled with neat cement grout, and the pocket was filled with concrete to the line, *Y-Y*, shown on the plans, leaving only the socket of the hinge exposed. After the concrete had become thoroughly set, the erection of the structural steel reinforcement was commenced. This was done before finishing the piers above the lines, *X-X*, in order to facilitate the handling of the erection equipment.

Two methods were used in erecting this structural reinforcement. Spans *C*, *D*, and the south arm of Span *A*, were raised with derricks and gin-poles, and without the use of falsework. For Span *B*, a two-bent timber gantry traveler was used. This traveler also raised the north arm of Span *A*. Span *D*, which is the shortest and lightest



CENTRE RIB DETAILS SPAN D
MAIN STREET VIADUCT
FORT WORTH, TEXAS

FIG. 8.



OUTER RIB DETAILS SPAN *D*
MAIN STREET VIADUCT
FORT WORTH, TEXAS.
FIG. 9.

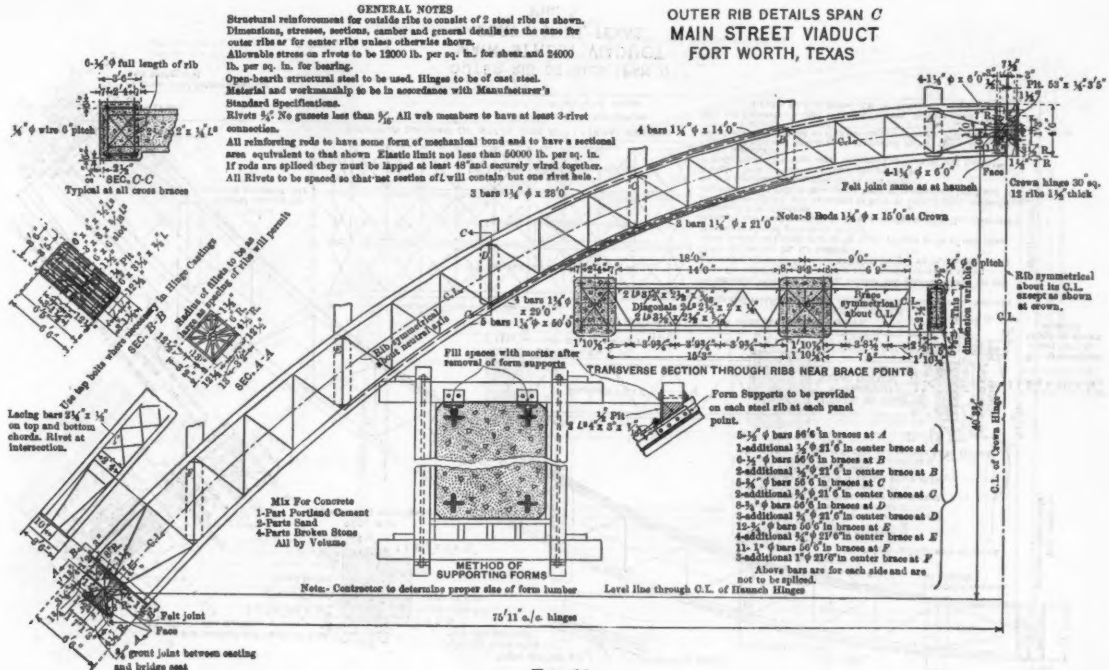

OUTER RIB DETAILS SPAN C
MAIN STREET VIADUCT
FORT WORTH, TEXAS
GENERAL NOTES
Structural reinforcement for outside ribs to consist of 2 steel ribs as shown.
Dimensions, stresses, sections, camber and general details are the same for outer ribs as for center ribs unless otherwise shown.
Allowable stress on rivets to be 12000 lb. per sq. in. for shear and 24000 lb. per sq. in. for bearing.
Open-hearth structural steel to be used. Hinges to be of cast steel.
Material and workmanship to be in accordance with Manufacturer's Standard Specifications.
Rivets 3/4". No gussets less than 5/16". All web members to have at least 3-rivet connection.
All reinforcing rods to have some form of mechanical bond and to have a sectional area equivalent to that shown. Elastic limit not less than 50000 lb. per sq. in.
If rods are spliced they must be lapped at least 48" and securely wired together.
All Rivets to be spaced so that net section of L will contain but one rivet hole.
6-1/2" ϕ full length of rib
1/8" ϕ wire 6" pitch
SEC. C-C
Typical at all cross braces
SEC. B-B
Use tap bolts where necessary in Hinge Castings
Radius of fillets to be as large as spacing of ribs will permit
SEC. A-A
Lacing bars 2 1/4" x 1/2" on top and bottom chords. Rivet at intersection.
Felt joint
Face
3/8" grout joint between casting and bridge seat
Rib symmetrical about neutral axis
C.L.
4 bars 1 1/4" ϕ x 14'0"
3 bars 1 1/4" ϕ x 28'0"
3 bars 1 1/4" ϕ x 21'0"
Note:-8 Rods 1 1/4" ϕ x 15'0" at Crown
3 bars 1 1/4" ϕ x 29'0"
5 bars 1 1/4" ϕ x 50'0"
4-1 1/4" ϕ x 6'0"
Felt joint same as at haunch
Crown hinge 30" sq. 12 ribs 1 1/2" thick
Plt. 53" x 1/4"-3'5"
Face
Rib symmetrical about its C.L. except as shown at crown.
C.L.
This dimension variable
Brace symmetrical about C.L.
Diagonals
TRANSVERSE SECTION THROUGH RIBS NEAR BRACE POINTS
Fill spaces with mortar after removal of form supports
Form Supports to be provided on each steel rib at each panel point.
METHOD OF SUPPORTING FORMS
Mix For Concrete
1-Part Portland Cement
2-Parts Sand
4-Parts Broken Stone
All by Volume
Note:- Contractor to determine proper size of form lumber
Level line through C.L. of Haunch Hinges
75'11" c./c. hinges
C.L. of Crown Hinge
40'3 1/2"
5-1/2" ϕ bars 56'6" in braces at A
1-additional 1/2" ϕ 21'6" in center brace at A
6-1/2" ϕ bars 56'6" in braces at B
2-additional 1/2" ϕ 21'6" in center brace at B
5-3/4" ϕ bars 56'6" in braces at C
2-additional 3/4" ϕ 21'6" in center brace at C
8-3/4" ϕ bars 56'6" in braces at D
3-additional 3/4" ϕ 21'6" in center brace at D
12-3/4" ϕ bars 56'6" in braces at E
4-additional 3/4" ϕ 21'6" in center brace at E
11- 1" ϕ bars 56'6" in braces at F
3-additional 1" ϕ 21'6" in center brace at F
Above bars are for each side and are not to be spliced.

FIG. 10.

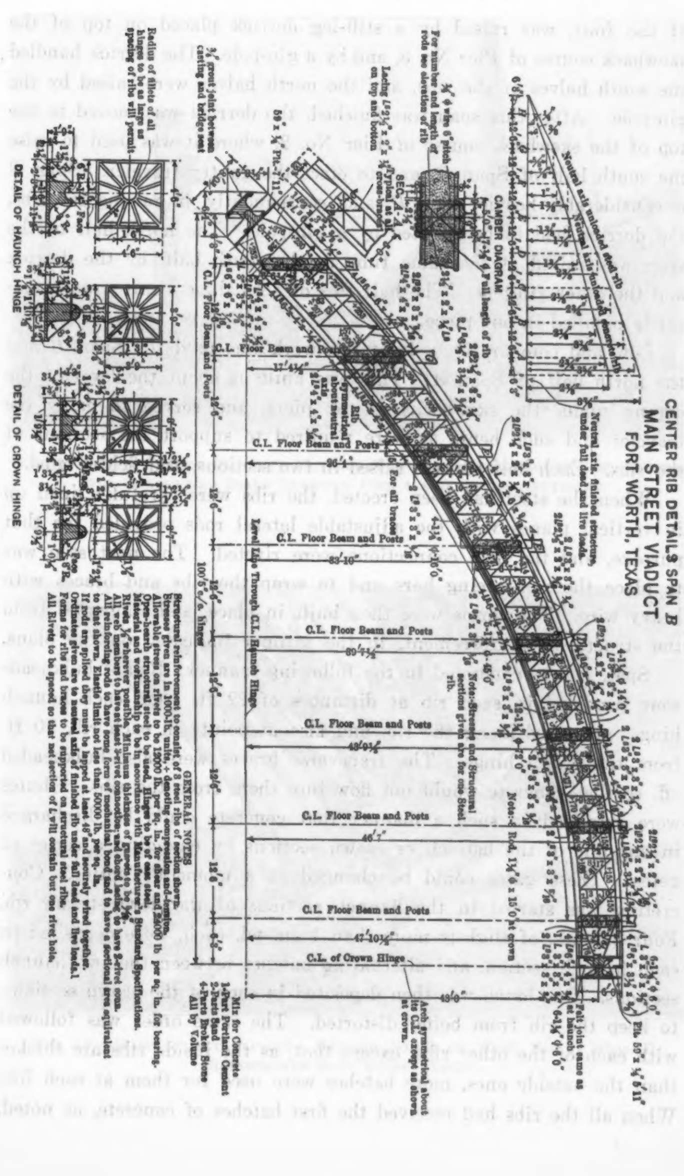

CENTER RIB DETAILS SPAN B
MAIN STREET VIADUCT
FORT WORTH, TEXAS
CAMBER DIAGRAM
C.L. Floor Beam and Posts
C.L. of Crown Hinge
GENERAL NOTES
Mix for Concrete
1-Part Portland Cement
2-Parts Sand
4-Parts Broken Stone
All by Volume
DETAIL OF HAUNCH HINGE
DETAIL OF CROWN HINGE

FIG. 11.

of the four, was raised by a stiff-leg derrick placed on top of the skewback course of Pier No. 3, and by a gin-pole. The derrick handled the south halves of the ribs, and the north halves were raised by the gin-pole. After this span was finished, the derrick was moved to the top of the skewback course of Pier No. 2, where it was used to raise the south half of Span *C*, and to erect the gantry traveler. Span *C* is considerably heavier than *D*, and, consequently, in raising the ribs, the derrick had to be assisted by a gin-pole. The north half of the span was raised at the same time as the south half, by the derrick and the other gin-pole. The half ribs of both these spans were necessarily handled in one piece.

Low and comparatively light falsework was used for Span *B* and the north half of Span *A*. This was built at about the level of the coping under the skewbacks of the piers, and served to carry the traveler and such bents as were required to support the sections of the ribs. Each half rib was raised in two sections, and field-riveted.

When the steel had been erected, the ribs were carefully lined up in vertical planes with the adjustable lateral rods provided for that purpose, and the field connections were riveted. The next step was to place the reinforcing bars and to wrap the ribs and braces with heavy wire. The forms were then built in place, and supported from the structural reinforcement, in the manner indicated on the plans.

Span *D* was concreted in the following manner: Radial bulkheads were provided in each rib at distances of 22 ft. from each haunch hinge, measured along the rib, and also at points in each rib, 10 ft. from the crown hinge. The transverse braces were also bulkheaded off, so that concrete would not flow into them from the ribs. Chutes were arranged in such a manner that concrete could be discharged in any one of the haunch or crown sections by changing a series of gates. These gates could be changed at a moment's notice. Concreting was started in the haunch sections of the down-stream rib. Four batches, of slightly more than 1 cu. yd. each, were deposited in each haunch section, and alternating batches between the two haunch sections. One batch was then deposited in each of the crown sections, to keep the rib from being distorted. The same order was followed with each of the other ribs, except that, as the inside ribs are thicker than the outside ones, more batches were used for them at each lift. When all the ribs had received the first batches of concrete, as noted,

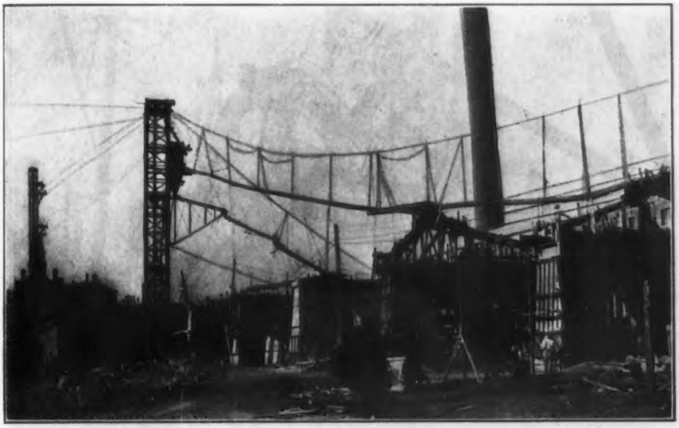


FIG. 12.—MAIN STREET VIADUCT: MOVABLE TOWER AND ARRANGEMENT OF CHUTES FOR CONCRETING PIERS BETWEEN PIER 4 AND NORTH ABUTMENT.

FIG. 13.—MAIN STREET VIADUCT: HAUNCH HINGE OF WEST RIB OF SPAN D. REINFORCING BARS BEING PLACED.

FIG. 14.—MAIN STREET VIADUCT: RAISING OF FIRST STRUCTURAL RIB. WEST RIB OF SPAN D RAISED WITH DERRICK AND GIN-POLE.

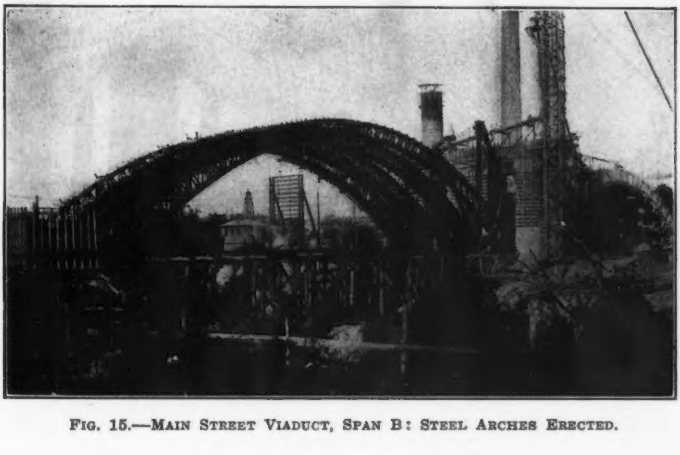


FIG. 15.—MAIN STREET VIADUCT, SPAN B: STEEL ARCHES ERECTED.

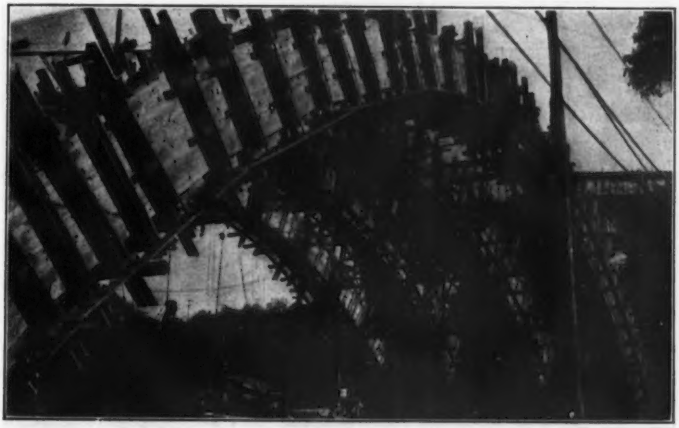


FIG. 16.—MAIN STREET VIADUCT, SPAN C: SHOWING RIBS COMPLETELY FORMED.

FIG. 17.—MAIN STREET VIADUCT: FORMING DECK OF SPAN C.

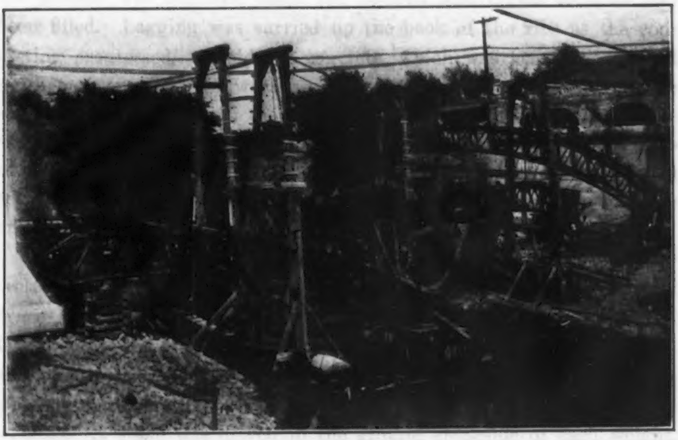


FIG. 18.—WEST SEVENTH STREET VIADUCT: RAISING EAST HALF OF SOUTH RIB OF ARCH BEFORE THE HIGH WATER.

FIG. 19.—ARCH RIBS AFTER THE WATER WENT DOWN, WEST SEVENTH STREET VIADUCT.

the process was repeated until the haunch and crown sections had been filled. Lagging was carried up the back of the ribs as the concreting progressed, to hold the concrete, which was necessarily mixed quite wet, in order to work well around the reinforcement. After setting for about 4 days, the bulkheads in the ribs were removed, the surface of the concrete was roughened up, and the closure sections of the ribs were concreted in about the same manner as the haunch sections. The transverse braces were not concreted until the rib concrete had set for several days.

In order to keep track of the distortion of the ribs while concreting was in progress, sash weights were suspended from the steel brace reinforcement close to each rib, at four points along the rib. One weight was placed at the brace on each side of the crown hinge, and one at each quarter point of the span. These weights were suspended by small wires, and carried scales which were wired to the weights. A stake was driven in the ground alongside of each weight, and a nail in the stake acted as a pointer on the scale. The initial reading was taken at each point, and another set of readings was made after each round of concreting. At first a slight settlement of the crown points was observed, followed by a slight rise. The distortion was less than had been anticipated, amounting to a maximum of about $\frac{1}{4}$ in.

The concrete in the ribs and braces was allowed to set for several days before the work of building the forms for the posts and deck of the span was begun. This portion of the work was concreted in the manner indicated under the head of "Loading". The posts were concreted, beginning at the haunches and working toward the crown. Then the deck was concreted from the crown back toward the haunches, with the exception of the middle section, over the crown, which was placed last, in order to provide for the expansion joint at the crown. The piers, in the meantime, had been finished to their full height, in order to insure stability and provide for the connection of the deck to the piers.

The concreting of the other spans was handled in practically the same manner as that just described. When the concreting of a section was started, it was carried through continuously, by working day and night if necessary, so that the work was made as nearly monolithic as possible.

The plant and equipment on the other viaducts was similar to that used at Main Street, but, of course, on a smaller scale. All four jobs were under construction at the same time, and were finished on or before the last of March, 1914.

Difficulties Encountered.—There was some difficulty, at Main Street, in the early stages of the work, in sinking the foundations of the main piers. Pier No. 1 gave the most trouble. The unstable character of the soil on the south side of the stream was responsible for this. Extra heavy timber sheet-piling and a large number of heavy shores of timber and steel were used. In order to give the material on the hillside south of the pier the least possible chance to slide, the excavation was made in four sections. The first section excavated was the down-stream quarter of the pier. This excavation was about 20 ft. in length, measured up and down stream. After rock was reached, the surface was carefully cleaned off and several drill holes were put down to test the rock. The surface of the rock was found to be very rough, with a slight drop in elevation away from the stream. The drill holes showed that the layers of rock were very thick and had only minute seams between them. There had been some fear that the pressure of the bank behind the pier might cause the pier to slide on the rock, before sufficient weight could be added to prevent it. With this in mind, preparations were made to cut a dowel in the rock so as to bond the pier and the rock together. The fear of sliding was dispelled when the extremely rough character of the rock was seen, and the dowel scheme was abandoned. The pier section was concreted up to the top of the 13 ft. 6-in. footing course, and the next section, which was the third quarter from the down-stream end, was excavated and concreted to the same height as the first. The up-stream quarter was next finished, and lastly the second quarter from the down-stream end. Of course, all longitudinal reinforcing bars in the portion of the pier below the top of the footing course, had to be spliced, on account of the method of construction adopted. Care was taken to bond the sections of the footing course together by vertical re-entrant joints extending the full height of the 13 ft. 6-in. course. Above the top of this course the pier was built as a monolith in the usual manner. In concreting the footing sections it was not thought advisable to remove all the shores as the concrete was carried up. Consequently, some of the steel members from the

Fig. 20.—Main Street Viaduct.

FIG. 21.—WEST SEVENTH STREET VIADUCT.

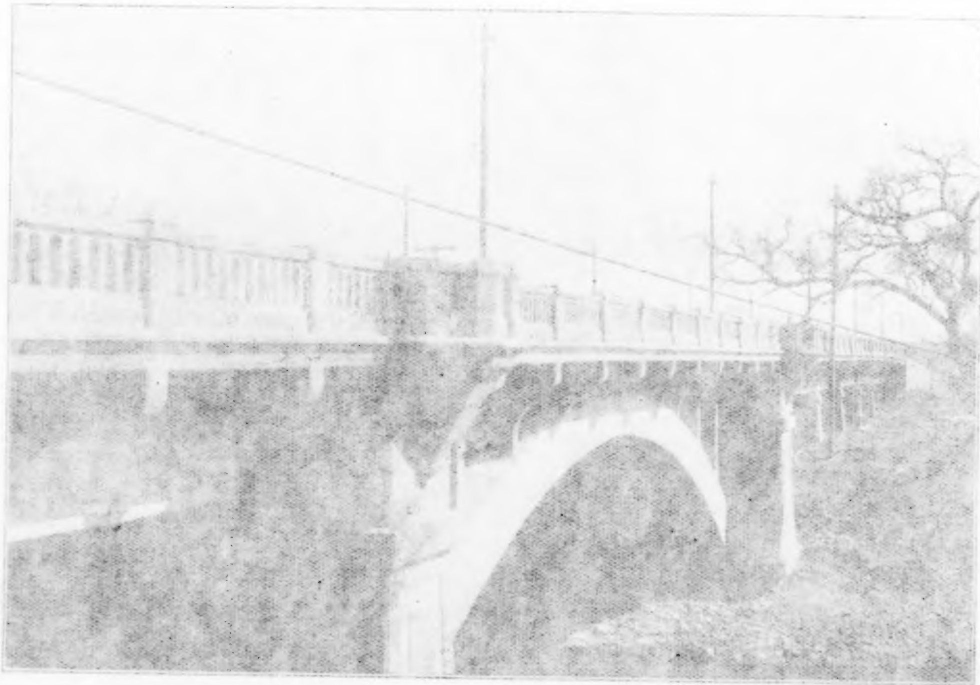


FIG. 21.—WEST SEVENTH STREET VIADUCT.

FIG. 22.—SAMUELS AVENUE VIADUCT.

old bridge, which had been used for shoring, were built into the pier and left.

At Pier No. 2 the only difficulty experienced was in driving the oak piles. These were about 20 ft. long, and were driven to refusal by using a drop-hammer. Driving through the compact mixture of sand and gravel at this point was difficult. A bearing test of the material in the bottom of the pit after the piles were driven gave very little settlement under a load of about 8 000 lb. per sq. ft., with a loaded area of about 4 sq. ft.

At West Seventh Street there was considerable trouble with the erection of the structural reinforcement of the arch span. The equipment was not as good as that used at Main Street, and the work progressed slowly. The engineers had expected that this steelwork would be handled by derricks on the piers in much the same manner as at Main Street. The sub-contractor felt that the size of the job did not warrant this equipment, and decided to crib up the steelwork on falsework in the bed of the stream. This was finally accomplished, and the falsework was removed, after the field splices in the ribs had been riveted. While the riveting of the transverse members was under way, the erector removed some of the lateral rods in order to get at the rivets with less trouble. This was done in spite of the protest of the engineers' representative. The stiffness of the lateral connections held the arch in place for a while, but finally a strong wind, aided to some extent by the movements of the men at work on the span, caused the ribs to revolve sideways on the hinges, and then drop into the stream a short distance below. Fortunately, no one was killed, and but slight damage was done to the steelwork. The work of re-erecting and repairing the steel was begun, the ribs being supported on timber falsework in the stream in much the same manner as before. When the work was only partly finished, the freshet previously mentioned, which occurred in September, 1913, came down and washed out the falsework, again dropping the steel into the river. The work was finally replaced in good shape, after much loss of time and money. This freshet also did some damage to the falsework for the traveler at Main Street. Had the arches been under construction on falsework in the usual manner, much more serious damage would undoubtedly have resulted.

Figs. 12 to 22 show the work at various stages, and are self-explanatory.

Cost of Viaducts.—Tables 1 to 5 give the quantities for each of the viaducts, and the unit prices bid by the successful contractor. They also give the cost per linear foot of each structure, the cost per square foot of horizontal and of vertical projection, and the cost per cubic foot. The cost per square foot of vertical projection is based on the area of projection of each structure between the finished

TABLE 1.—COST OF MAIN STREET VIADUCT.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	4 720 cu. yd.	$0.35	$1 652.00	0.43
2	Foundation excavation	15 227 " "	1.00	15 227.00	3.96
3	Rock excavation	444 " "	2.00	888.00	0.23
4	(*a*) Concrete No. 1 (1:2:4)	10 611 " "	10.25	108 762.75	28.16
	(*b*) Concrete No. 2 (1:2½:5)	14 880 " "	6.85	101 928.00	26.40
	(*c*) Concrete No. 3 (1:3:6)	438 " "	6.25	2 737.50	0.71
5	Railings	3 875 lin. ft.	2.00	7 750.00	2.01
6	Structural steelwork	1 537 400 lb.	0.05	76 870.00	19.90
7	Steel reinforcing bars	1 375 150 "	0.035	48 130.25	12.46
8	Steel castings	205 460 "	0.07	14 382.20	3.72
9	Iron castings	11 173 "	0.04	446.92	0.11
10	Anchor-bolts and T. P. casings	21 311 "	0.06	1 278.66	0.33
11	Steel dowels	98	3.00	294.00	0.08
13	Rip-rap	836 cu. yd.	1.50	1 254.00	0.32
14	Manholes	2	50.00	100.00	0.03
15	Removing old bridge	....		2 500.00	0.65
17	Timber piles	194	10.00	1 940.00	0.50
	Total			$386 141.28	100.00

TABLE 2.—COST OF WEST SEVENTH STREET VIADUCT.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	5 424 cu. yd.	$0.25	$1 356.00	1.22
2	Foundation excavation	5 570 " "	0.61	3 397.70	3.07
3	Rock excavation	132 " "	3.00	396.00	0.36
4	(*a*) Concrete No. 1 (1:2:4)	2 890.4 " "	11.82	34 164.53	30.90
	(*b*) Concrete No. 2 (1:2½:5)	5 138 " "	7.25	37 250.50	33.69
	(*c*) Concrete No. 3 (1:3:6)	440 " "	6.15	2 706.00	2.45
5	Railings	2 120 lin. ft.	2.25	4 770.00	4.31
6	Structural steelwork	196 975 lb.	0.0475	9 356.31	8.46
7	Steel reinforcing bars	389 089 "	0.029	11 283.58	10.21
8	Steel castings	45 628 "	0.058	2 646.42	2.39
9	Iron castings	4 470 "	0.03	134.10	0.12
10	Anchor-bolts and T. P. casings	7 063 "	0.047	331.96	0.30
11	Steel dowels	30	1.50	45.00	0.04
12	Sidewalks on earth fill	5 240 sq. ft.	0.21	1 100.40	1.00
13	Rip-rap	363 cu. yd.	2.00	726.00	0.66
14	Manholes	4	38.00	152.00	0.14
15	Removing old bridge			750.00	0.68
	Total			$110 566.50	100.00

TABLE 3.—Cost of Samuels Avenue Viaduct.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading....................	1 111 cu. yd.	$0.30	$333.30	0.58
2	Foundation excavation........	2 739.6 " "	1.00	2 739.60	4.78
3	(a) Concrete No. 1 (1 : 2 : 4)....	1 237.2 " "	12.00	14 846.40	25.91
	(b) Concrete No. 2 (1 : 2½ : 5)..	3 413.6 " "	7.50	25 602.00	44.68
4	Railings....................	906 lin. ft.	2.00	1 812.00	3.16
5	Structural steelwork..........	12 739 lb.	0.05	636.95	1.11
6	Steel reinforcing bars.........	247 755 "	0.03	7 432.65	12.97
7	Iron castings.................	3 420 "	0.04	136.80	0.24
8	Anchor-bolts and T. P. casings.	6 388 "	0.06	383.28	0.67
9	Rip-rap....................	690.25 cu. yd.	2.00	1 380.50	2.41
10	Timber piles................	200	10.00	2 000.00	3.49
	Total..........................			$57 303.48	100.00

TABLE 4.—Cost of East Fourth Street Viaduct.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading....................	4 107 cu. yd.	$0.40	$1 642.80	3.11
2	Foundation excavation........	2 832 " "	1.00	2 832.00	5.37
3	(a) Concrete No. 1 (1 : 2 : 4)...	1 236 " "	12.00	14 832.00	28.11
	(b) Concrete No. 2 (1 : 2½ : 5)..	2 734.5 " "	7.50	20 508.75	38.87
4	Railings....................	905 lin. ft.	2.00	1 810.00	3.43
5	Structural steelwork..........	12 739 lb.	0.05	636.95	1.20
6	Steel reinforcing bars.........	238 061 "	0.03	7 141.83	13.54
7	Iron castings.................	3 420 "	0.04	136.80	0.26
8	Anchor-bolts and T. P. casings.	6 388 "	0.06	383.28	0.73
9	Rip-rap....................	907 cu. yd.	2.00	1 814.00	3.44
10	Timber piles................	82	10.00	820.00	1.56
11	Removing existing piers......	....	......	200.00	0.38
	Total..........................			$52 758.41	100.00

TABLE 5.—Summary of Cost of Four Viaducts.

	Main Street.	West Seventh Street.	Samuels Avenue.	East Fourth Street.
Cost per linear foot of structure....	$244.60	$123.45	$141.29	$130.59
Cost per square foot of horizontal projection..................	3.66	2.10	3.28	3.02
Cost per square foot of vertical projection..................	6.34	6.93	3.53	3.60
Cost per cubic foot of volume between finished ground line and crown of roadway..............	0.091	0.119	0.082	0.083
Total estimated cost, including paving, lighting, and engineers' fees..	$428 882.00	$127 472.00	$63 584.00	$58 766.00

ground line and the top of the roadway. For the horizontal projection, the extreme width over the copings or stringer mouldings was taken. In computing the cost per cubic foot, the volume included between vertical planes through the copings, and between the finished ground line and the top of the roadway, was taken. The total, on which the above unit costs are based, is the cost of the structure based on the contractor's unit prices and the quantities as given in the tables, to which has been added the cost of the paving, lighting, and engineers' fees.

Personnel.—The work was designed and its execution supervised by Messrs. Brenneke and Fay, Consulting Engineers, of St. Louis, Mo. The writer, as Principal Assistant Engineer for that firm, was responsible for the design of the viaducts, and exercised executive supervision over the work. L. H. Faidley, Jun. Am. Soc. C. E., had charge of the detailed work of preparing the plans, and William Holden, Jun. Am. Soc. C. E., was in direct charge as Resident Engineer.

The contract for the Main Street Viaduct was awarded to the Hannan-Hickey Brothers Construction Company, of St. Louis. The Tarrant Construction Company, of Fort Worth, had the contract for the West Seventh Street Viaduct, and the other two structures were awarded to the McKensie-Williams Construction Company, of Webb City, Mo. The Virginia Bridge and Iron Company, of Roanoke, Va., were the sub-contractors for the structural steelwork of the Main and West Seventh Street structures, and erected the steelwork at Main Street. Austin Brothers, of Dallas, Tex., were the sub-contractors for the erection of the steelwork at West Seventh Street.

In conclusion, the writer wishes to express his thanks to Mr. Faidley, and to Mr. Holden for valuable assistance in the preparation of this paper.

DISCUSSION

Mr. Gayler.

CARL GAYLER,* M. AM. SOC. C. E. (by letter).—It is not often that important structures such as these concrete viaducts are described as fully and clearly as in this paper. Publications of this kind are of great value to the Profession.

Reinforced concrete viaducts of the general layout and appearance of the four Fort Worth viaducts are not uncommon, but the construction of arches by using steel centers embedded in the concrete of the arches and forming an integral part of them is of recent date.

At first glance, comparison with the Melan arch suggests itself; there are, however, the following radical differences between the two systems:

1. The steel ribs of the Melan arch are not designed to be self-sustaining over the full length of the span under the weight of the concrete of the arch proper.

2. The usual assumption for the computation of Melan arches—that the concrete resists the direct thrust and the steel ribs take up the bending moment—is, considering the distance between the steel ribs, open to criticism. On the other hand, the suppositions under which the arches of the Fort Worth viaducts were calculated are definite and logical, and may be summarized as follows: The steel ribs alone carry, in the first stage of the work, their own weight, that of the concrete as it is being poured into the forms around them, and that of the lateral braces. The stresses thus produced in the steelwork can be accurately ascertained; the concrete is merely dead weight, and the question of the difference of the moduli of elasticity of the two materials is not involved. The final stresses in the arch ribs resulting from the additional weight of the superstructure, and from the live loads, are provided for as in other reinforced concrete arches, the steelwork of the ribs forming part of the steel reinforcement, as fully explained by Mr. Bowen.

3. The distribution of the metal through the body of the concrete in the ribs of the Fort Worth arches is far superior to that in any Melan arch ever built.

After careful consideration of the problem with which the consulting engineers had to deal, that is, to build, safely and economically, the arches of the viaducts under the difficult conditions of a limited clearance, the probability of sudden and extraordinary floods, and an unstable river bed, the writer is convinced that the plan adopted was the one logical solution of the problem. Under analogous conditions, it will undoubtedly be repeated.

The conclusion, however, which Mr. Bowen has reached, that "for high structures, and for those over streams subject to sudden and

* St. Louis, Mo.

Mr. Gayler. great variation of water level, this method is cheaper and safer than to use falsework supported from the bed of the stream", needs modification, inasmuch as, in the case of high structures, this method has to compete, not so much with ordinary falsework, as with steel centers placed below the concrete.

Where the height of the arch above the high-water line admits of the use of "free" centers, that is, centers under the concrete, the latter method, in most cases, will be found to be the more economical.

In the first place, the height of the steel centers is then independent of the thickness of the concrete arches or arch ribs, and great saving in the weight of the steel, especially in long spans, can be obtained by choosing an economical height for the steel trusses or arches.

It is also feasible to get along with a set of centers for one (longitudinal) half of the arch only, and to utilize this set again for the other half. On a number of concrete viaducts great saving in the cost of erection has been obtained in thus using the falsework or centering over again, as for instance, on the great viaduct built a few years ago in the Duchy of Luxemburg, on the Walnut Lane Bridge, at Philadelphia, Pa., the Rock River Bridge, at Cleveland, Ohio, and the Kings Highway Viaduct, at St. Louis, Mo. Even the scrap value of the free centers is an item worth taking into account.

The saving in steel gained by utilizing the steel ribs as reinforcement of the arch will thus in most cases be greatly overbalanced if free centers are used. The arches of numerous concrete viaducts have been proportioned so that bending moments were practically eliminated and, of course, in all such cases no strengthening of the arch by the embedded steel ribs would be claimed.

Another advantage of free centers is that they are applicable to the circular as well as to the ribbed arch. The method used on the Fort Worth arches, except for short spans, is probably limited to the ribbed arch, on account of its greater available thickness.

An excellent illustration of the advantages of free centers over embedded centers, in the case of a high structure, is found in the successful erection of the Rock River Bridge, at Cleveland, Ohio, by three-hinged steel centers, designed by Wilbur J. Watson, M. Am. Soc. C. E.*

Mr. Bowen states that the object of his paper is "to bring out a discussion of the type of reinforced concrete arch in which the reinforcement is composed of structural shapes and is designed to support the weight of the forms of plastic concrete in the arch ring on ribs", but a few remarks on other features of the viaducts may be allowed.

In looking over the foundations of the piers, as shown on Plates XVIII, XIX, and XX, an impression was left on the writer's mind that the old conflict between the sincere endeavor of the engineer

* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 1.

to obtain the best results and the barrier set up by a limited appropriation has not been decided fully in favor of the engineer, as far as the question of foundations of the piers of the Fort Worth bridges is concerned. This is written with considerable diffidence, because a thorough acquaintance with the nature of the material in the river bed alone could justify criticism. The following points, therefore, are brought out merely as suggestions, and will be touched on as briefly as possible. Mr. Gayler.

The footings of a number of the piers in the bed of the river, that is, inside of the levee protections, for instance, Pier 3 of the Main Street Viaduct, the piers of the Seventh Street Viaduct with the exception of Nos. 1 and 2, and some of the piers of the East Fourth Street Viaduct, are not carried down as deep as the river bed. For their protection against scouring, rip-rapping is relied on, which may, or may not, be properly renewed in years to come.

The base of Pier 2 of the West Seventh Street Viaduct, judging by the small-scale elevation on Plate XVIII, seems so provokingly close to rock that it may not be unreasonable to ask why this chance to obtain an unyielding foundation on both sides of a rather flat concrete arch was not utilized.

Even the problem of a solid foundation for Pier 2 of the Main Street Viaduct, carefully as it seems to have been studied by the consulting engineers, leaves some doubt in the mind whether rock might not have been reached at a reasonable price. According to Plates XVIII and XX, the 15-ft. oak piles at the north side of the pier reach down to within a few feet of it.

As to the design of the bearings of the track rails, adopted for reasons of economy, it is not likely to be repeated on other viaducts. The life of the fiber pad, ½ in. thick, which is to act as a shock absorber, will not be a long one, and the hammering of the wheels under the electric railway traffic will then be a serious matter for the slabs and girders.

In speaking of the advantages of the three-hinged, ribbed arch, it was an oversight on Mr. Bowen's part to claim that water-proofing is, in every case, a necessity for the circular arch; but, aside from this, every word he has said is true, and should be welcomed by every structural engineer. Concrete arches should either be built with hinges or efficiently reinforced against temperature stresses. Too long, for the credit of the Profession, have concrete arches been built in the United States with disregard for temperature stresses or under assumptions of changes of temperature which, on their face, are not true.

Changes of temperature in the body of concrete arches produced by long-continued heat or cold can be ascertained, either by judgment confirmed by experience, or by actual tests on existing structures.

Mr. Gayler. Whenever excessive stresses result from such changes, hinges should be used.

As the writer expressed it in a previous publication: This (that is, the assumption of inadequate changes of temperature in order to obtain convenient stresses in the arch) is the only instance on record in the history of engineering where an attempt is made to adjust the laws of Nature to the works of man, instead of *vice versa*.

It is true that in former years the three-hinged steel arch had gradually to give way to the two-hinged or to the fixed arch, on account of its lack of rigidity under moving loads, but with the heavy concrete arch this defect is, to a great extent, overcome, and should not be a serious objection to the use of the center hinge.

Mr. Bowen. S. W. BOWEN,* M. AM. SOC. C. E. (by letter).—Referring to the use of "free" centers mentioned by Mr. Gayler, these were considered for the Fort Worth viaducts, but were not adopted for the following reasons: At West Seventh Street, there was not sufficient clearance between the high-water line and the underside of the arch rib to permit of their use. At Main Street, all the spans were of different lengths, which, at most, would have permitted the use of the centers only twice, once for each half of the structure. The ribs required considerable permanent reinforcement, so that in this case it was more economical to place the centers within the ribs, and use them both as centers and reinforcement.

"Free" centers have been used to great advantage in a number of large viaducts where there were several spans of the same length, and where little or no reinforcement was required in the arch, for example, for the "Tunkhannock", and other viaducts, built by the Delaware, Lackawanna and Western Railroad Company. The first use made of these centers in the United States was for the Rocky River Bridge, at Cleveland, Ohio, mentioned by Mr. Gayler.

Pier 2, at West Seventh Street, is founded on a very dense mass of gravel and boulders. It was originally intended to place it on bed-rock, but, after an examination of the foundation, it was decided to stop at the gravel formation. The other piers were carried down to such a depth that, considering the ample bank protection, it seems highly improbable that there will be any trouble from scour. The reason for stopping the piles under Pier 2 at Main Street short of bed-rock is explained in the paper.

The arrangement for fastening the track rails did not prove to be as satisfactory as had been expected, principally on account of the difficulty of setting the anchor-bolts. This scheme was used in order to reduce the dead load on the structure.

In conclusion, the writer wishes to express his thanks to Mr. Gayler for his valuable discussion.

* St. Louis, Mo.

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Paper No. 1330

PROOF OF AN ASSUMPTION IN THE THEORY OF CONCRETE BEAMS

By Ralph E. Goodwin, Jun. Am. Soc. C. E.

With Discussion by J. P. J. Williams, Assoc. M. Am. Soc. C. E.

Synopsis.

The object of this paper is to present a mathematical justification of the theory that a section of a beam which is a plane before bending remains a plane after bending, as applied to beams of reinforced concrete.

The proof is based on the principle of least work.

The assumptions are those usual in the theory of reinforced concrete design, with the exception of that of a plane before and after bending, which is assumed to be true of the compression side of the beam, but not of the beam as a whole.

It is important to notice that the value of the conclusions depends on the accuracy of the assumptions. No assumptions are made in the proof, however, which are not commonly used in reinforced concrete design.

The common theory of reinforced concrete beam design is based on the assumption that a section which is a plane before bending remains a plane after bending. This assumption is borrowed from the common theory of flexure, and is so familiar to engineers, through its application to beams of homogeneous material, that the correctness of applying

it to beams of reinforced concrete is seldom questioned. Nevertheless, the difference between a beam of homogeneous material and a composite beam of concrete reinforced with steel is so great that an assumption which is reasonable in the case of the former is not necessarily reasonable in the case of the latter, and textbooks on the subject should furnish more plausible reasons for the theory of a plane before and after bending than they usually advance.

There follows a mathematical justification of the principle in question, which is based on the principle of least work. It is assumed that the correct value of j (arm of resisting couple) is the value which develops any given M (resisting moment) with the minimum of work expended in bending the beam. The method of proof used is to compute the value of j by the method of least work and to show that the value thus determined is the same as that in common use.

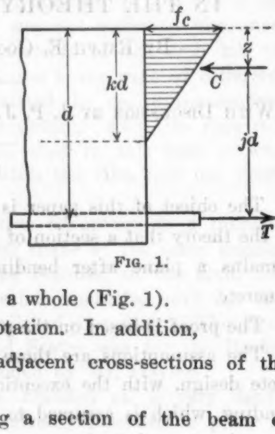

FIG. 1.

In the work which follows, the assumptions usual to the theory of reinforced concrete design are made, with the exception of that of a plane before and after bending, which is assumed to be true of the compression side of the beam, but not of the beam as a whole (Fig. 1).

The symbols used are the standard notation. In addition,

dl = the distance between two adjacent cross-sections of the beam before bending.

W_{dl} = the work done in straining a section of the beam of length, dl.

W_{dl} = (work of straining steel on tension side) plus (work of straining concrete on compression side).

According to the principles of mechanics, the work done by a load gradually applied is one-half the force multiplied by the distance.

Hence,

$$W_{dl} = \frac{1}{2} C \frac{2}{3} \frac{f_c}{E_c} \times dl + \frac{1}{2} T \frac{f_s}{E_s} \times dl$$

$$= \frac{1}{2} \left(\frac{f_c\, b\, k\, d}{2} \right) \frac{2}{3} \frac{f_c}{E_c} \times dl + \frac{1}{2} (A\, f_s) \frac{f_s}{E_s} \times dl$$

$$W_{dl} = \frac{f_c^2\, b\, k\, d}{6\, E_c} \times dl + \frac{f_s^2\, A}{2\, E_s} \times dl \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

$$M = \frac{f_c\, b\, k\, d}{2} \times j\, d = f_s\, A\, j\, d\,;$$

$$f_c = \frac{2\, M}{b\, k\, j\, d^2} = \frac{2\, M}{3\, b\, (1-j)\, j\, d^2};\quad f_s = \frac{M}{A\, j\, d}.$$

Substitute these values in Equation (1).

$$W_{dl} = \left(\frac{2\, M}{3\, b\, (1-j)\, j\, d^2}\right)^2 \frac{3\, b\, (1-j)\, d}{6\, E_c} \times dl + \frac{M^2}{2\, A\, j^2\, d^2\, E_s} \times dl$$

$$= \frac{M^2}{2\, d^3\, b\, E_c} \left\{ \frac{4}{9\, (1-j)\, j^2} + \frac{1}{p\, n\, j^2} \right\} dl$$

$$= \frac{M^2}{2\, d^3\, b\, E_c} \left\{ \frac{4\, (j^2 - j^3)^{-1}}{9} + \frac{j^{-2}}{p\, n} \right\} dl.$$

To find the value of j which will make W_{dl} a minimum, set the first derivative with respect to j equal to zero.

$$\frac{d\, (W_{dl})}{d\, j} = 0 = \frac{M^2}{2\, d^3\, b\, E_c} \left\{ \frac{-4\, (j^2 - j^3)^{-2}\, (2\, j - 3\, j^2)}{9} - \frac{2\, j^{-3}}{p\, n} \right\} dl;$$

$$0 = \frac{4\, (2\, j - 3\, j^2)}{9\, (j^2 - j^3)^2} + \frac{2}{p\, n\, j^3};$$

$$0 = \frac{1}{j^3\, (1-j)^2} \left\{ 4\, p\, n - 6\, j\, p\, n + 9\, (1-j)^2 \right\}.$$

Each factor, when set equal to zero, gives a root of the equation. The root desired lies in the last factor,

$$0 = 4\, p\, n - 6\, j\, p\, n + 9\, (1-j)^2;$$

$$0 = j^2 - \frac{2}{3}\, j\, (p\, n + 3) + (1 + \frac{4}{9}\, p\, n).$$

Solving for j by completing the square in the above equation gives,

$$j = 1 - \frac{1}{3}\left(\sqrt{2\, p\, n + (p\, n)^2} - p\, n\right) = 1 - \frac{1}{3}\, k.$$

Hence $\quad k = \sqrt{2\, p\, n + (p\, n)^2} - p\, n.$

This is the value of k in common use. *Q. E. D.*

For beams reinforced for compression and for T-beams the demonstration is similar to the foregoing, up to and including the placing

of the first derivative equal to zero. From that point the general solution becomes so complicated that it is best to continue as follows,

(1) Assume numerical values for p, p', d, d', n;

(2) Compute the numerical value of k from the formula in common use;

(3) Substitute the foregoing values in the following equation:

First derivative of work with respect to $k = 0$.

It will be found that the equation in (3) is always satisfied by the values assumed in (1) and (2). *Q. E. D.*

DISCUSSION

J. P. J. Williams,* Assoc. M. Am. Soc. C. E. (by letter).—This mathematical proof of the validity of the usual assumption in reinforced concrete beam analysis, that plane sections remain plane after flexure, that is, that the steel strain is just sufficient to make this assumption correct, is very interesting. The almost universal respect accorded the method of least work should insure that this new application will strengthen materially the belief in the validity of the assumption discussed. It should be emphasized that this proof is also based on the assumption that the compression side remains plane, that is, the usual straight line variation for compressive stress is assumed. Hence the proof really applies to the tensile steel, assuming no tension in concrete, and shows that the work done by the steel to produce a minimum total work is that resulting from a strain just sufficient to keep the normal section a plane. Mr. Williams.

The writer finds that the analysis for T-beams can be made without much difficulty, if the variable be made z instead of k. In fact, this method of analysis is also much shorter for the case of the rectangular beam treated by the author.

Both these analyses follow, using the author's notation.

Rectangular Beam.

Write the equation for work, W_{dl}, in terms of z, and finally solve for $k = \frac{3z}{d}$.

$$W_{dl} = \frac{1}{2} C \cdot \frac{2}{3} \frac{f_c}{E_c} dl + \frac{1}{2} T \frac{f_s}{E_s} dl.$$

Substitute $f_c = \frac{2C}{b\,kd} = \frac{2C}{3bz}$; $f_s = \frac{T}{p\,bd}$; $E_s = n\,E_c$

$$W_{dl} = \frac{dl}{E_c\,b} \left[\frac{2\,C^2}{9\,z} + \frac{T^2}{2\,pn\,d}\right].$$

Substitute $C = T = \frac{M}{d - z}$

$$W_{dl} = \frac{M^2\,dl}{E_c\,b\,(d-z)^2} \left[\frac{2}{9\,z} + \frac{1}{2\,pn\,d}\right].$$

Place the first derivative of the variables with respect to z equal to zero:

$$\frac{d}{dz} \left\{ (d-z)^{-2} \left[\frac{2\,z^{-1}}{9} + \frac{1}{2\,pn\,d}\right] \right\} = 0.$$

* New York City.

Mr. Williams.

Or, $-2\ (d-z)^{-3}\ (-1)\left[\frac{2\,z^{-1}}{9}+\frac{1}{2\,pn\,d}\right]+(d-z)^{-2}$

$$\left(-\frac{2\,z^{-2}}{9}\right)=0.$$

Multiply through by $9\,z^2\,(d-z)^3\,\frac{pn}{d}$:

$$4\,z\,\frac{pn}{d}+\frac{9\,z^2}{d^2}-\frac{2\,pn}{d}\,(d-z)=0.$$

Solve for $\frac{3\,z}{d}=k$, by collecting terms, and completing square:

$$\left(\frac{3\,z}{d}\right)^2+6\,pn\,\frac{z}{d}+(pn)^2=2\,pn+(pn)^2$$

$\frac{3\,z}{d}=k=\sqrt{2\,pn+(pn)^2}-pn$, as found by the author.

T-Beam, Neglecting Compression in Stem.

$$W_{dl}=\frac{1}{2}\,C\times\text{strain at }z\text{ from top}+\frac{1}{2}\,T\,\frac{f_s}{E_s}\,dl.$$

Strain at z from top $=\frac{f_c}{E_c}\,\frac{kd-z}{kd}\,dl.$

Also, substitute $f_s=\frac{T}{A}$; $E_s=n\,E_c$; $f_c=\frac{2\,kd\,C}{b\,t\,(2\,kd-t)}$.

$$W_{dl}=\frac{dl}{E_c}\left[\frac{C^2\,(kd-z)}{bt\,(2\,kd-t)}+\frac{T^2}{2\,n\,A}\right].$$

Substitute $C=T=\frac{M}{d-z}$.

$$W_{dl}=\frac{M^2\,dl}{E_c\,(d-z)^2}\left[\frac{kd-z}{bt\,(2\,kd-t)}+\frac{1}{2\,n\,A}\right].$$

Now, $z=\frac{3\,kd-2\,t}{2\,kd-t}\cdot\frac{t}{3}$, or, $kd=\frac{3\,t\,z-2\,t^2}{6\,z-3\,t}$,

and $kd-z=\frac{6\,t\,z-2\,t^2-6\,z^2}{6\,z-3\,t}$

$$2\,kd-t=\frac{-t^2}{6\,z-3\,t}.$$

Substitute, and place first derivative with respect to z equal to zero: Mr. Williams.

$$\frac{d}{d\,z}\left[(d-z)^{-2}\left(\frac{-6\,t\,z+2\,t^2+6\,z^2}{b\,t^3}+\frac{1}{2\,n\,A}\right)\right]=0.$$

$$-2\,(d-z)^{-3}\,(-1)\left(\frac{6\,z^2-6\,t\,z+2\,t^2}{bt^3}+\frac{1}{2\,n\,A}\right)$$

$$+\frac{(d-z)^{-2}}{b\,t^3}\,(12\,z-6\,t)=0.$$

Multiply through by $(d-z)^3\,b\,t^3$:

$$2\,(6\,z^2-6\,t\,z+2\,t^2)+\frac{b\,t^3}{n\,A}+(d-z)\,(12\,z-6\,t)=0.$$

$$12\,z^2-12\,t\,z+4\,t^2+\frac{b\,t^3}{n\,A}+12\,dz-6\,d\,t-12\,z^2+6\,t\,z=0.$$

$$6\,z\,(2\,d-t)=6\,d\,t-4\,t^2-\frac{b\,t^3}{n\,A}.$$

$$z=\frac{2\,n\,A\,(3\,d-2\,t)-b\,t^2}{2\,n\,A\,(2\,d-t)}\cdot\frac{t}{3}.$$

This is exactly the same as the value obtained by the usual assumption, as can be shown by substituting the value of $kd=\dfrac{2\,n\,d\,A+b\,t^2}{2\,n\,A+2\,b\,t}$ in the expression for z:

$$z=\frac{3\,k\,d-2\,t}{2\,k\,d-t}\cdot\frac{t}{3}=\frac{6\,n\,d\,A+3\,b\,t^2-4\,n\,t\,A-4\,b\,t^2}{4\,n\,d\,A+2\,b\,t^2-2\,n\,t\,A-2\,b\,t^2}\cdot\frac{t}{3}.$$

Or, $$z=\frac{2\,n\,A\,(3\,d-2\,t)-b\,t^2}{2\,n\,A\,(2\,d-t)}\cdot\frac{t}{3},$$ as before.

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Paper No. 1331

INVESTIGATION OF THE PERFORMANCE OF A REACTION TURBINE

BY R. L. DAUGHERTY, ESQ.*

WITH DISCUSSION BY MESSRS. S. J. ZOWSKI, H. BIRCHARD TAYLOR, DANIEL W. MEAD, R. C. CARPENTER, AND R. L. DAUGHERTY.

SYNOPSIS.

A careful and accurate test of a 550-h.p. reaction turbine recently installed at Cornell University showed a very high efficiency. In addition to the regular load test at constant speed, the torque, head, and discharge were measured at zero speed, and readings were also taken of speed, head, and discharge under no load. From these data it was possible to construct, at least approximately, a complete characteristic curve for the turbine.

1.—Cornell University generates its power for lighting and other purposes in a hydro-electric power plant which operates under a head of approximately 140 ft. The water is led from Beebe Lake to the power-house through a pipe line 5 ft. in diameter and 1 711 ft. long. When the present power-house was built, in 1905, it was equipped with four Pelton impulse wheels, but the increasing demands for power have made it necessary to add to the capacity of the plant. After due consideration of the various factors involved, it was decided to put in a reaction turbine for the additional unit.†

* Assistant Professor of Hydraulics, Sibley College, Cornell University.

† "Choice of Turbine for University Power House", by R. L. Daugherty, *Sibley Journal of Engineering*, Vol. 28, p. 191.

2.—The turbine selected was made by the I. P. Morris Company, of Philadelphia. It is rated at 550 h.p., at 600 rev. per min., under a head of 142 ft. The runner is 27 in. in diameter. The turbine is direct-connected to a 450-kv-a., 2 300-volt, 3-phase, 60-cycle alternator, supplied by the General Electric Company.

3.—The new turbine was tested by the writer in February, 1914. In this work he was ably assisted by Mr. F. G. Switzer, a graduate student and Fellow in Sibley College. The latter calibrated all the electrical instruments used and took charge of the determination of the generator output and efficiency.

4.—With the data secured in this test, it was possible to construct curves which would show the operation of the turbine under any combination of head, power, and speed. This is rarely done except in the case of a brake test. On account of the favorable conditions for testing, it is believed that a higher degree of accuracy than usual was attained. The efficiency of the turbine itself was found to be very high.

5.—The method of estimating the head in a turbine test is a matter of agreement in any case. The head consumed by the wheel alone is the fall from the surface of the head-water to the surface of the tail-water, minus all losses in the penstock and draft-tube. It is usually obtained by taking the difference in the heads at the entrance to and exit from the turbine. Let the pressure, when reduced to feet of water, be denoted by p, the elevation in feet above any arbitrary datum plane by z, and the velocity, in feet per second, as determined by the cross-sectional area and the measured rate of discharge, by v. If subscript ($_1$) be used to indicate values at the point of entrance to the turbine case and subscript ($_2$) corresponding values at the point of discharge from the turbine into the draft-tube, the head on the turbine will be:

$$h = z_1 + \frac{v_1^2}{2g} + p_1 - z_2 - \frac{v_2^2}{2g} - p_2.$$

6.—The draft-tube is often regarded as an integral part of the turbine, and thus any losses connected with it are to be charged to the turbine. Thus, the head on the turbine and draft-tube will be the fall from the head-water to the tail-water, minus the loss in the penstock

only; or, it is obtained in a test, if z_1 is the elevation of the point where the pressure is measured above the tail-race, by

$$h = z_1 + \frac{v_1^2}{2\,g} + p_1.$$

This value of h will be greater than that given by the equation in Paragraph 5. The difference between the two is equal to the friction head in the draft-tube plus the velocity head lost at the discharge from the mouth of the draft-tube. To illustrate this it might be noted that for Run 7 the head computed by the method shown in Paragraph 5 was 137.9 ft., and, as computed by the method just given, it was 140.46 ft. Throughout this paper the latter method will be used.

7.—In the test the pressure at the intake was measured with a mercury column built in the form of a U-tube. The height of the mercury columns was read on a metal scale graduated to hundredths of a foot. The oscillation of the columns was usually less than 0.010 ft., so that accurate readings were readily secured. There were four pressure connections equally spaced around the circumference of the penstock, and the true pressure was taken as the average of the four readings. It was noted, however, that there was very little difference in the pressures as measured at the four points.

8.—For convenience, the pressure was transferred to the center of the shaft which was 3.82 ft. above the zero point on the metal scale. If the two mercury column readings be denoted by R_1 and R_2, the pressure transferred to the center of the shaft will be

$$13.57\ (R_1 - R_2) - (3.82 - R_2).$$

The vertical distance from the center of the shaft to the crest of the weir was 14.0 ft. If the head of water over the weir be noted by H, the elevation of the center of the shaft above the tail-race level will be

$$14.0 - H.$$

The sum of these two expressions will be the value of

$$z_1 + p_1.$$

The intake pipe at the point where the pressure was measured is 30 in. in diameter. From this area and the measured rate of discharge, the value of v_1 may be obtained. By adding $\frac{v_1^2}{2\,g}$ to the above, we get the value of the head on the turbine.

FIG. 1.—CORNELL UNIVERSITY POWER-HOUSE.

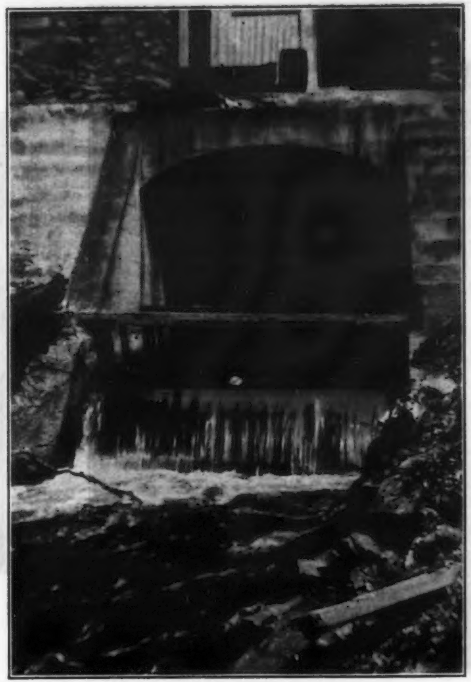

FIG. 2.—WEIR IN TAIL-RACE.

FIG. 1.—[illegible]

FIG. 2.—[illegible]

FIG. 3.—REACTION TURBINE, WITH DRAFT-ELBOW REMOVED, SHOWING RUNNER.

FIG. 4.—WATER RHEOSTAT.

9.—The rate of discharge was measured with a suppressed weir in the tail-race. A steel plate with a sharp beveled edge in good condition was set in just before the test. The length of the crest was 7.573 ft., and its height above the bottom of the channel was 2.83 ft. A set of baffles was inserted just below the draft-tube, and a raft was floated on the water to kill the surface ripples. The head on the weir was measured with a hook-gauge. The zero was set by a surveyor's level and checked by an observation when the water was just ready to flow over the weir. At the conclusion of the test, the zero was again checked by the level. During the test there was a small constant leakage through the other wheels. This was determined by a leakage run before the test was begun.

10.—The rate of discharge was computed by the modified Francis formula:

$$q = 3.33\,L\left(H + \frac{v^2}{2\,g}\right)^{\frac{3}{2}}$$

where,

q = cubic feet per second;

L = length of crest, in feet;

H = head on weir, in feet; and

v = velocity of approach, in feet per second.

If the velocity of approach is uniform over the entire cross-section of the channel, this formula is reasonably correct. In many cases, however, the actual velocity near the surface is much greater than the mean velocity, and there is practically a dead-water space near the bottom of the channel. Thus the effective velocity head is greater than that determined from the mean velocity. As v is obtained by dividing the rate of discharge by the cross-sectional area of the channel, it is customary to multiply $\frac{v^2}{2\,g}$ by a factor which may range from 1.0 to 2.0, according to circumstances. In this case a traverse of the channel with a current meter, by E. W. Schoder, Assoc. M. Am. Soc. C. E., and Professor Turner, showed a uniform distribution of velocity, and thus justified the use of a factor of unity.

11.—This uniform distribution of velocity is probably due to the fact that the distance from the draft-tube to the crest of the weir is only about 30 ft. The draft-tube is vertical, and discharges its water

directly against the bottom, thus it is probable that there would be a high bottom velocity for some distance. If the tail-race had been much longer we might then have expected to find the more usual (that is, unequal) distribution of velocities.

12.—The power output of the generator was absorbed by a water rheostat suspended in a pool in the creek. This rheostat consisted of three pipes at the vertices of an equilateral triangle, with sheet-iron plates bolted to them. The load was varied by changing the depth of immersion. A similar rheostat, used in some testing in 1913, proved to be very satisfactory, giving a perfectly steady load. The one used in this test was designed to carry twice as much load as the former instrument, but it failed to do so without modification. The design of such a rheostat seems to be largely a matter of trial, in view of the limited data available and also on account of the important factor of

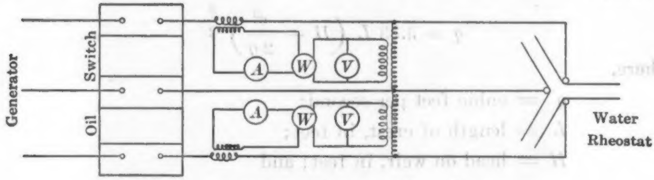

DIAGRAM OF CONNECTIONS

FIG. 5.

the purity of the water.* At the time of the tests in 1914, the run-off in the creek was mostly from melting ice and snow and therefore purer than the ground-water encountered in 1913.

13.—In order to make the rheostat carry the load, it was necessary to bend the plates so as to bring them within 2 in. of each other. It is probable that the fluctuations on the heavier loads were due to the boiling of the water between the plates. By taking readings every 2 min. during quite a period of time, it is believed that a good average was obtained, despite this slight unsteadiness. The accuracy of the results is roughly checked by the fact that the power factor remained approximately constant for all the loads.

14.—All electrical instruments used were carefully calibrated by comparison with standards in the possession of the Physics Department of Cornell University. The ratios of current and voltage trans-

* See *Power*, August 4th, 1914, p. 184.

formers were determined, as well as the phase angles of the latter. The phase angles of the current transformers were found to be negligible.

15.—At the end of each run the resistance of the armature was determined by the drop-of-potential method. The friction, windage, and iron losses were found by operating the generator as a synchronous motor. It was also desired to find the value of the friction and windage alone, and to determine how these losses varied with the speed. As this could not be done with the synchronous motor run, a small, direct-current motor was belted to the set. The input to the motor was read, and the motor losses were deducted. If the power of this motor had been sufficient to drive the set at 600 rev. per min.,

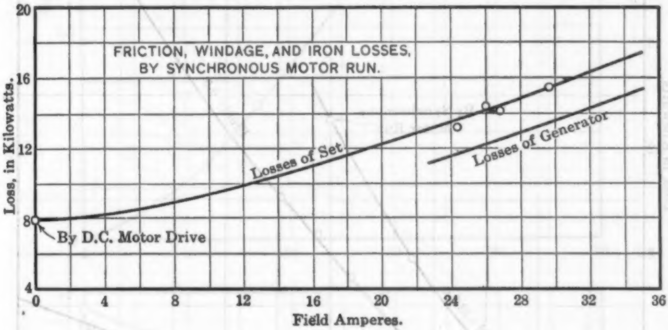

FIG. 6.

under full field, the synchronous motor run would have been unnecessary. The two combined, however, gave everything that was desired. (Figs. 6 and 7.) The friction and windage of the generator was arbitrarily assumed to be three-fourths of that of the set.

16.—In the test, runs were made under various loads at a constant speed of 600 rev. per min. The turbine was also prevented from rotating, and the torque at various gate openings was measured by a system of levers and standard weights. The gate-openings were the same as those used in the load runs. Readings were also taken of head and discharge. In addition, the turbine was permitted to run at run-away speed, the only load on it being the friction and windage of the generator. For the same gate openings, readings were taken of speed, head, and discharge.

17.—The usual commercial test, made at a constant speed and under an approximately constant head, is quite valuable, yet it is not entirely satisfactory. The question is always open as to what the effect would have been if the speed had been a little higher or a little lower. Also, in many plants, where the speed of the turbine must necessarily be constant, the head will vary through quite a range, due

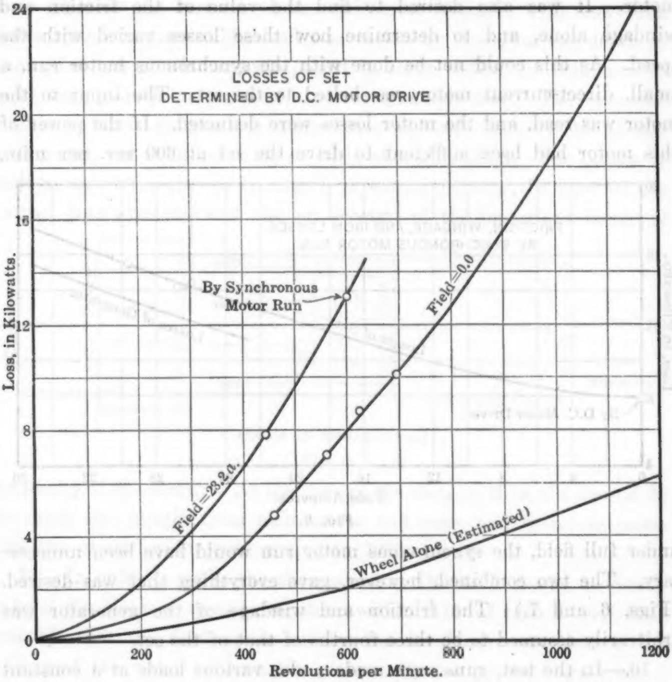

Fig. 7.

to fluctuations in the stream flow. Thus, in actual operation the relation between speed and head may not always be the best. In order to determine fully the characteristics of a given turbine, it is necessary to test it at various gate openings under several relations of speed to head. It is clear that the speed may be varied, as in the usual brake test, or it may be possible to vary the head. Either method will accomplish the desired result. In the test under discussion the head

was throttled down by the gate-valve to about two-thirds of its normal value. A load run was then made under the regular speed of 600 rev. per min. It was hoped to secure a number of such points at several

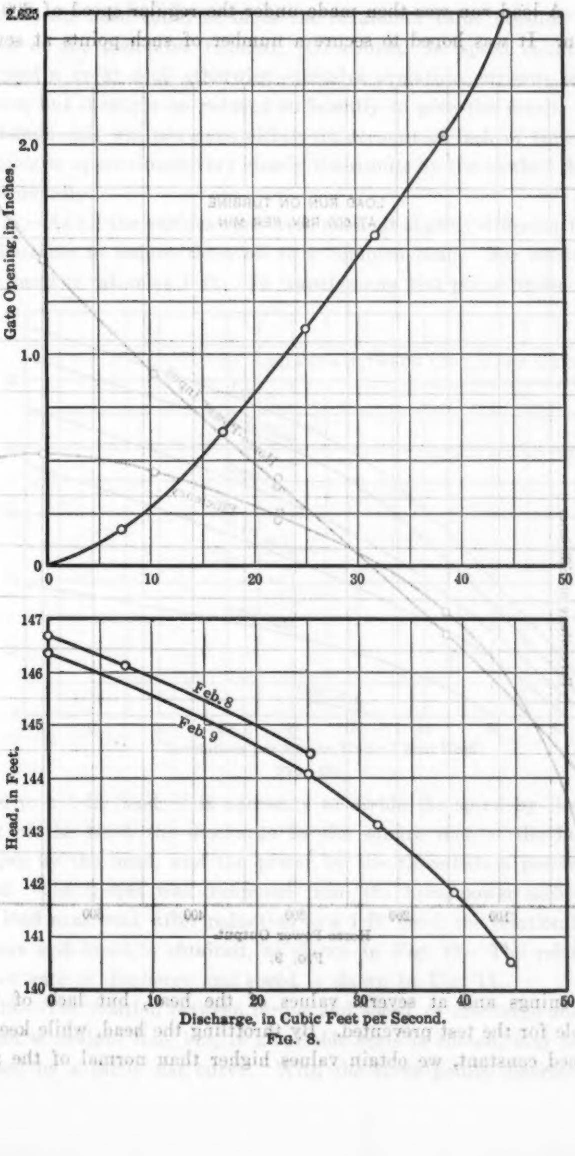

FIG. 8.

gate-openings and at several values of the head, but lack of time available for the test prevented. By throttling the head, while keeping the speed constant, we obtain values higher than normal of the ratio

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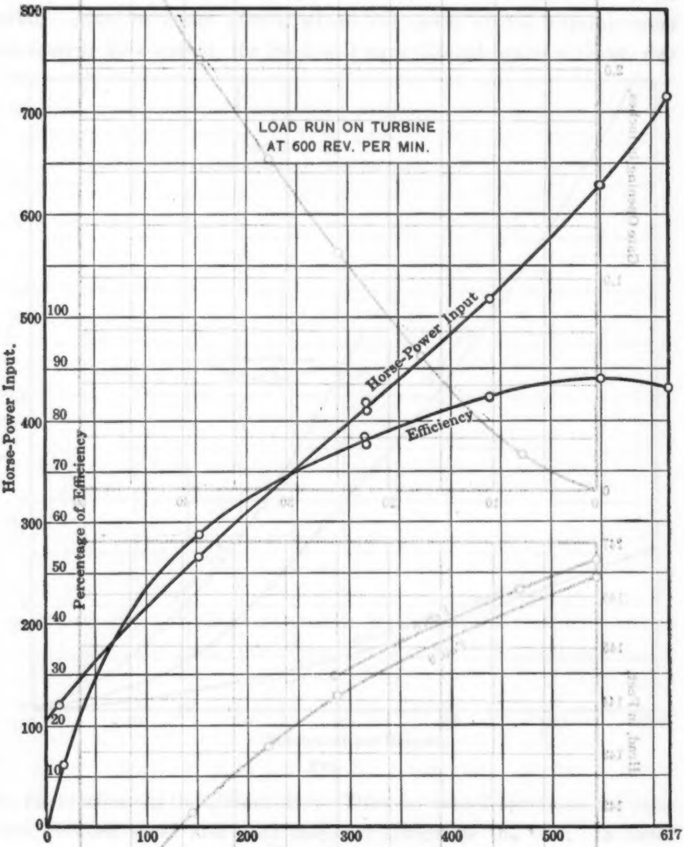

FIG. 9.

gate-openings and at several values of the head, but lack of time available for the test prevented. By throttling the head, while keeping the speed constant, we obtain values higher than normal of the ratio

of speed to head. In order to obtain values of this ratio lower than normal, the head could be made the maximum value possible and the speed of the generator reduced. Of course, the speed could not be reduced a great deal, otherwise excessive armature currents would be drawn, but it might be reduced sufficiently to give the results desired. Although this was not done either, on account of lack of time, it was possible to approximate very closely the results by the method described subsequently.

18.—As all the various runs were made at slightly different heads, it is desirable to reduce them all to a common head. For convenience, this head is taken as 1 ft. To transfer any test point under a given

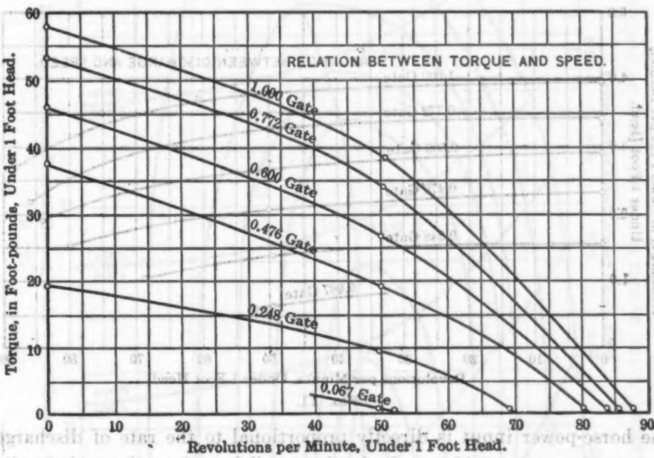

FIG. 10.

head to a 1-ft. head, it is necessary to divide the speed by the square root of the head, the discharge by the square root of the head, the torque by the head, and the power by the three-halves power of the head. The torque was computed from the horse-power measured in the load runs, and, after reduction to a 1-ft. head, the relation between torque and speed is obtained, as shown in Fig. 10. The relation between rate of discharge and speed is shown in Fig. 11.

19.—The relation between torque and speed, or discharge and speed, is not a straight line, but, if a suitable scale is chosen, may be represented by a fairly flat curve. With the three points determined for

each gate opening, it is possible to draw curves which cannot be very far from correct, assuming that the three points are accurately determined. If additional points had been determined by the methods previously outlined, there would then have been no question about the entire curve; but these additional points are not essential. These three points, moreover, cover the range from zero speed to the maximum run-away speed under no load.

20.—All these relations may be involved in a set of curves in a diagram which is known as the characteristic curve of the turbine. To construct such a diagram all values must be reduced to a 1-ft. head. The co-ordinates are rate of discharge and revolutions per minute. As

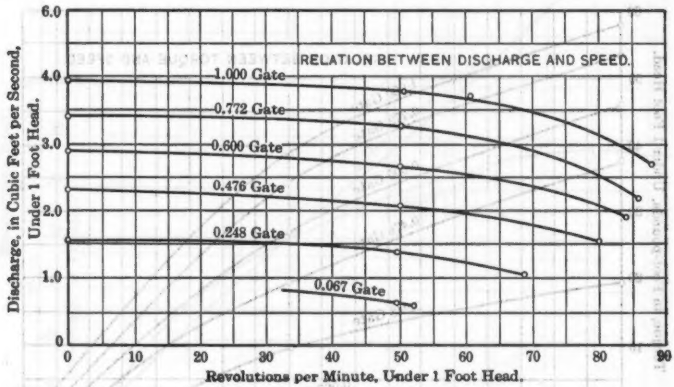

FIG. 11.

the horse-power input is directly proportional to the rate of discharge, it is customary to add a scale for it, parallel to the values of the discharge. Within this diagram, curves are constructed for the various gate openings, showing the relation between discharge and speed. The diagram is completed by the construction of iso-efficiency curves. Occasionally, iso-power curves are also added, but they tend to make the diagram too complicated, and are really not necessary. For any conditions of operation, the horse-power input and the efficiency may be readily noted on the diagram. The product of the two gives the horse-power developed. This system of plotting a turbine performance was devised by D. W. Mead, M. Am. Soc. C. E., and is given in his "Water Power Engineering".

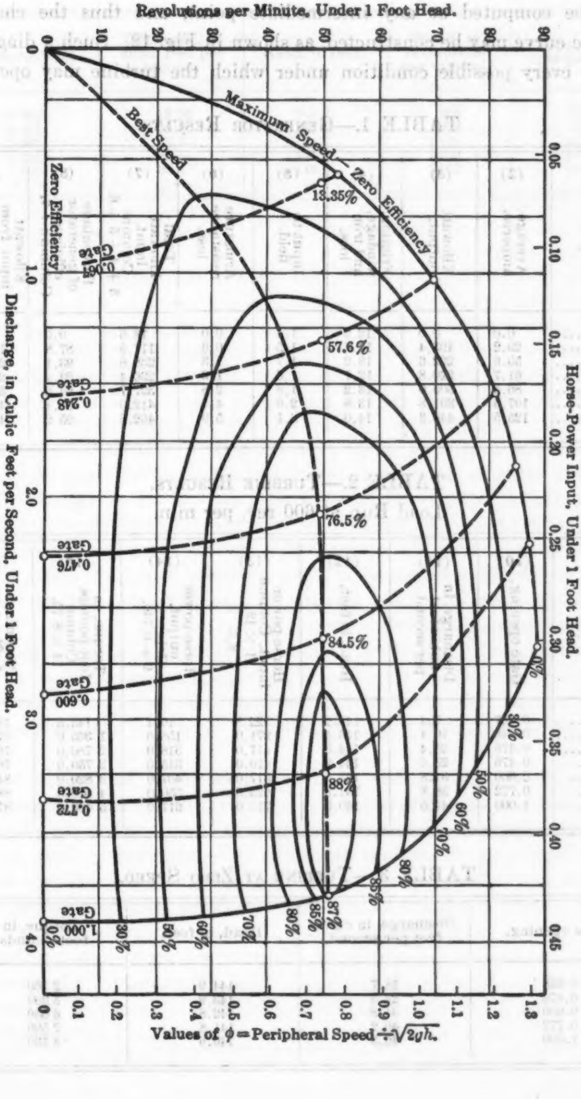

FIG. 12.

CHARACTERISTIC CURVE OF 27-INCH I. P. MORRIS TURBINE, TESTED UNDER 142 FEET HEAD.

21.—Accepting the curves in Figs. 10 and 11 as correct, the efficiency may be computed at any intermediate point, and thus the characteristic curve may be constructed, as shown in Fig. 12. Such a diagram covers every possible condition under which the turbine may operate.

TABLE 1.—Generator Results.

(1) Run.	(2) Average amperes.	(3) Kilowatt output.	(4) Friction, windage, and iron loss.	(5) Input to field.	(6) Armature resistance loss.	(7) Total kilowatt input. Columns 3 + 4 + 5 + 6.	(8) Percentage of generator efficiency. Column 3 ÷ 7.	(9) Kilowatt input from turbine. Columns 3 + 4 + 6.
1...........	0.0	0.0	12.2	1.4	0.0	13.6	0.0	12.2
2...........	29.2	103.4	12.6	1.5	0.3	117.8	87.8	116.3
3...........	59.6	222.6	13.2	1.7	1.3	238.8	93.1	237.1
4...........	61.7	220.8	12.7	1.6	1.3	236.4	93.4	234.8
5...........	86.5	309.9	13.2	1.8	2.8	327.7	94.6	325.9
6...........	107.6	391.8	13.8	2.0	4.4	412.0	95.1	410.0
7...........	125.5	440.3	14.0	2.1	5.9	462.3	95.2	460.2

TABLE 2.—Turbine Results.
Load Run at 600 rev. per min.

(1) Run.	(10) Gate opening.	(11) Discharge, in cubic feet per second.	(12) Head, in feet.	(13) Horse-power input. Column $\frac{11 \times 12}{8.8}$.	(14) Horse-power output. Column 9 ÷ 0.746.	(15) Torque, in foot-pounds. Column 14 × 8.75.	(16) Percentage of efficiency. Column 14 ÷ 13.
1...........	0.067	7.4	146.1	122.8	16.4	143.5	13.35
2...........	0.248	16.4	145.4	271.0	156.0	1 365.0	57.6
3...........	0.476	25.4	144.3	417.0	318.0	2 780.0	76.3
4...........	0.476	25.0	144.2	410.0	315.0	2 750.0	76.7
5...........	0.600	31.8	143.1	517.0	437.0	3 820.0	84.5
6...........	0.772	38.8	141.8	625.0	550.0	4 820.0	88.0
7...........	1.000	44.5	140.5	710.0	617.0	5 400.0	87.0

TABLE 3.—Turbine at Zero Speed.

Gate opening.	Discharge, in cubic feet per second.	Head, in feet.	Torque, in foot-pounds.
0.248	18.7	144.9	2 760
0.476	27.5	143.9	5 390
0.600	34.8	142.8	6 550
0.772	40.2	141.8	7 520
1.000	46.3	140.6	8 130

TABLE 4.—TURBINE AT RUN-AWAY SPEED.

Gate opening.	Discharge, in cubic feet per second.	Head, in feet.	Speed, in revolutions per minute.	Torque, in foot-pounds.
0.248	12.55	145.7	835	90
0.476	18.8	145.2	969	98
0.600	23.0	144.7	1 010	101
0.772	26.8	144.0	1 029	103
1.000	32.4	143.4	1 052	104

DISCUSSION

Mr. Zowski.

S. J. ZOWSKI,* ESQ. (by letter).—Regarding the question of head for which a water turbine is responsible: In Paragraph 5 the author gives a formula which, as a close examination will show, defines the turbine head as the vertical distance from the level in the forebay, at the entry into the pipe line, to that in the tail-race, around the draft-tube, minus the friction head of the pipe line, minus the friction head in the draft-tube, minus the final discharge velocity head.

In Paragraph 6 he gives a formula which defines the turbine head as the same vertical distance between the two levels, minus the friction head in the pipe line.

This latter definition of the net head is that which has been used almost invariably, and the writer does not see any sufficient reason for abandoning it, at least, not when the definition of head serves the purpose of settling the question of efficiency guaranties.

The first definition eliminates the friction and final discharge loss in the draft-tube, thus not charging these losses to the turbine. This is an unwarranted—if not directly incorrect—method of procedure. The draft-tube is an essential part of the water turbine, its function being to make good in what the runner fails and frequently must fail. It utilizes or recovers that amount of energy which the runner does not, or, owing to some necessary limitations in its design, cannot utilize. With some wheels this amount is necessarily relatively large, so that the efficiencies of these turbines would not have acceptable values were it not for the use of a well-proportioned draft-tube. The draft-tube, therefore, is essential as a part of the prime mover, and, consequently, whatever losses are obtained therein must be charged to the whole turbine. This explains why turbine builders and designers always insist that the draft-tube be either designed by them, or, at least, that its design, if made by others, shall be approved by them.

The elimination of the draft-tube loss would be parallel to the elimination of the area below the exhaust line of an indicator card of a steam engine. Nobody would approve of that, when the efficiency of the engine is determined from the indicator cards. Only in one instance might there be reason for the elimination of draft-tube losses, namely, when comparisons are made between turbines which have had to receive draft-tubes of widely differing proportions.

When, however, the matter of efficiency guaranty is being decided or agreed on, there is no excuse for this elimination. The designer and builder, if he sees that local conditions over which he has no control will not allow him to put in a draft-tube of the proportions

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which he would like to use, will take this into account accordingly, and make an allowance in his guaranty. Mr. Zowski.

Mr. Taylor.

H. BIRCHARD TAYLOR,* ESQ. (by letter).—This paper should prove of considerable interest. The securing of an efficiency as high as 88% in a turbine of such small dimensions and of such low power and specific speed, is unusual. In the last few years we have become accustomed to efficiencies of more than 90% at the Holyoke testing flume, and results in excess of this in turbines of high power and of large dimensions, after installation. The dimensions of a turbine, as well as its power output, have a considerable bearing on the maximum efficiency which may be secured. For turbines of homologous design. the unit having the largest dimensions will develop the highest efficiency.

In the writer's experience, the Cornell turbine gives the highest efficiency ever secured in an authenticated test of a unit of such small dimensions and capacity. The testing of the turbine at different speeds, and the relations found between torque and discharge at various speeds, and at zero speed, are of considerable interest.

In the design of the Cornell turbine, the best proportions could not be used in every detail and, on account of utilizing existing patterns, the value of ϕ was made abnormally large. This probably accounts for the part-gate efficiency not being as high as the excellent value of maximum efficiency would lead us to expect. In such small units, the design cannot be made geometrically similar in all respects to a large unit of the same specific speed. Fewer buckets must be used in the runner in order to permit wider openings for the passage of trash; and, for mechanical reasons, a lower number of guide-vanes must be used, and the vanes are thicker in proportion. Therefore, the slight modifications in the design, which have been necessary because of the small size of the unit, would probably lower the part-gate efficiencies to a certain extent.

The writer had an opportunity of examining the test data under discussion previous to the publication of the paper, and, with the exception of the head measurement, he is in entire agreement with the methods used. Some correspondence has passed between the author and the writer regarding head measurement, and, as there is still a disagreement over this point, the writer thought that in his discussion it would be advisable to confine his remarks entirely to this question, especially as there may be differences of opinion among engineers as to just what constitutes the correct effective head on a turbine. The author has very kindly given the writer permission to quote passages from his letters.

* Hydr. Engr., I. P. Morris Co., Philadelphia, Pa.

Mr. Taylor.

The method of measuring the head, as specified in the contract for this turbine, is as follows:

"In determining the efficiency of the unit, the head shall be measured in the following manner: four mercury gauges shall be attached to the penstock near the intake flange of the casing, the gauges being equally spaced around the circumference. The average reading of these gauges, reduced to feet of water, when added to the vertical distance between the height of the mercury in the pressure side of the gauges and the level of the tail-water at the discharge from the draft-tube, shall be considered the head, after being corrected for the difference in velocity at the point where the gauges are attached and at the discharge from the draft-tube."

The writer contends that this is the correct method of measuring the head, as it charges against the turbine all losses due to friction, eddies, whirls, etc., from the beginning of the turbine (the intake flange of the casing), to the end of the turbine (the end of the draft-tube). It does not, however, nor should it, charge against the turbine energy which would not be effective in the turbine, even should the efficiency of the latter equal 100%, such as the losses external to the turbine, or which occur outside of the limits specified.

There is no disagreement with the author over the question as to whether a turbine should be charged with the losses through the racks, intake, and penstock, for all agree that it is the efficiency of the turbine which is to be determined and not the over-all efficiency of the plant, which is a different matter. The disagreement arises solely from the question as to whether the turbine should be charged with the energy represented in the velocity at the exit from the draft-tube.

Three or four years ago this question was frequently discussed with engineers of power companies during the preparation of contracts. In the past three years, however, there has been no disagreement between the various power companies and the writer in regard to the measurement of the effective head on the turbine. In every contract it has been stated clearly that the velocity head at the exit from the draft-tube will not be charged against the turbine.

In Paragraph 5 of the paper the author states that the method of estimating the head in a turbine test is a matter of agreement in any case. This is perfectly true, but there is no reason that the method agreed on cannot represent the true effective head.

The following are quotations from one or two of the author's letters regarding this question:

"You will note that in my paper I computed the head as I did in my report of the test, and not as specified in the contract. Some correspondence passed between us on this subject and I have since taken the question up with several others and find my contention borne out by them.

"In my letters I stated the question thus: 'Is it more logical to use (6) [method outlined in Paragraph 6] or to subtract from that the velocity head at exit from the draft-tube'. Mr. Taylor.

"One reply received is as follows: (*a*) 'I have your letter and wish to say that the logical and customary way of measuring the head under which a turbine is acting is that mentioned under (6)'." [That is, Paragraph (6), without this velocity head correction.]

The only case where the velocity head from the draft-tube into the tail-race could be deducted would be when making comparisons of two turbines in which, owing to local conditions, this discharge speed had to be made widely different.

"From the Holyoke Water Power Company I received: (*b*) 'We would say that practically all our tests are made in open flume, and that the head is always taken to be the actual distance between the water surfaces in the flume and in the tail-race. We have never made any deductions whatever from this observed head. From our view, the draft-tube is a necessary adjunct to the wheel, and the velocity head therein is directly chargeable to the wheel'.

"In addition, I received a verbal comment to the same effect from Professor R. C. Carpenter.

"You understand that I do not maintain that your position is entirely wrong, for I see the merits of the case where it comes to comparing two turbines under widely different settings. But why not eliminate the draft-tube altogether, as in Paragraph (5) of my paper? I should like to have you bring up this point also [in the discussion]. My view is that you are proceeding only half-way when you wish to leave out the velocity head from the draft-tube."

Referring to this last quotation, namely, the elimination of the draft-tube entirely, it is possible to give the author's views on this matter a little more clearly by quoting from a letter from him dated November 9th, 1914, as follows:

"I admit that there is merit in your contention for the reason stated by you and the correspondent quoted [see Paragraph (*a*) preceding]. That is to say, the same turbine set in the two different ways shown would give different efficiencies due to the superior draft-tube in the one to the left [referring here to Fig. 13].

"Yet the difference is not in the turbines themselves, but in their draft-tubes. It is not a question even of draft-tube design, but is simply one of setting. But it seems to me that the only true way to compare turbines is to eliminate the draft-tube altogether. Thus we would say

$$h = z_1 + \frac{v_1^2}{2g} + p_1 - z_2 - \frac{v_2^2}{2g} - p_2.$$

"In that event, our efficiency would be the same for both. The difference between this and $h_1 = z_1 + \frac{v_1^2}{2g} + p_1$ is equal to the friction loss in the draft-tube plus the discharge loss, $\frac{v_3^2}{2g}$. If you sub-

Mr. Taylor. tract the velocity head only for each one of the figures [Figs. 13 and 14], you do not subtract as much for the left one as for the right-hand one. (Yet, due to the greater length and, possibly, greater leakage due to greater suction, the losses in the left suction tube (2 to 3) are greater than the right-hand one.) That would make the turbine on the left appear to give a slightly lower efficiency than the one on the right, unless the draft-tube losses were deducted also. The subtraction of $\frac{v_3^2}{2g}$ from the head at (1) is only a partial step toward the true result needed for comparison."

In reference to the quotation (*a*) previously given, the writer disagrees with the author of the statement. The charging of the exit velocity from the draft-tube against the efficiency of the turbine is neither logical nor customary. It may be the method used by the party quoted,

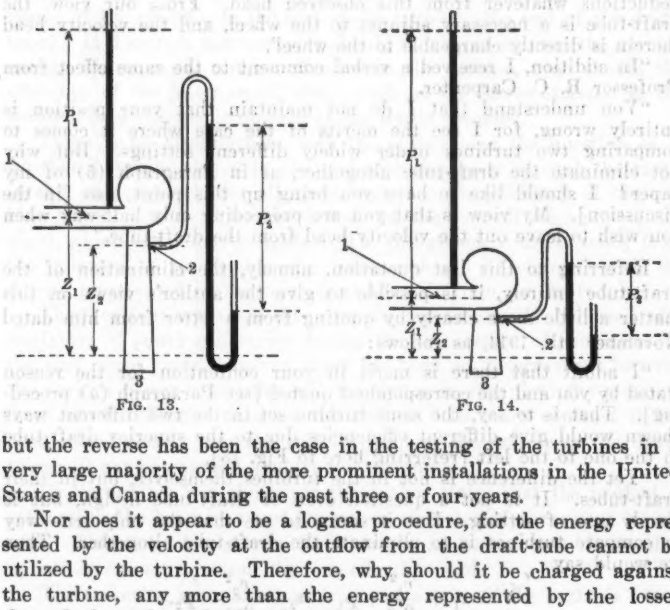

FIG. 13. FIG. 14.

but the reverse has been the case in the testing of the turbines in a very large majority of the more prominent installations in the United States and Canada during the past three or four years.

Nor does it appear to be a logical procedure, for the energy represented by the velocity at the outflow from the draft-tube cannot be utilized by the turbine. Therefore, why should it be charged against the turbine, any more than the energy represented by the losses through the racks or in the penstock?

It may be argued that it is within the province of the turbine builders to control the magnitude of this loss, and that it can be kept within negligible values by a proper design of the draft-tube; also, as the turbine builders are responsible for the design of the draft-tube, this loss should be charged against the turbine. In answer to this, it

may be stated that, in the writer's experience, he has often been restricted in the dimensions of the outflow sections of the draft-tube, due very often to the desire of the power companies to keep the dimensions of the power-house as small as possible, and also to avoid rock excavation. It cannot be said that the discharge area of the draft-tube can be fixed by the turbine builder in every case. Mr. Taylor.

It must be borne in mind that in first-class installations the velocity head at the discharge from the draft-tube does not exceed from one-quarter to three-quarters of 1% of the total head acting on the turbine, so that it would seem on first thought that the question is not a serious one. The writer, however, will endeavor to point out that there are serious aspects to this question of head measurement which deserve the careful consideration of hydraulic engineers.

Referring to the quotation (*b*), previously given, from the Holyoke Water Power Company: This does not answer the author's question. Because the Holyoke Water Power Company has used, for a great number of years, a certain method of head measurement, it does not follow that it is the correct one, and it cannot be said that the methods used at the Holyoke flume are by any means scientific.

Referring again to the same quotation, the statement that "the draft-tube is a necessary adjunct to the wheel, and the velocity head therein is directly chargeable to the wheel" is misleading, and does not answer the author's question.

The draft-tube is not only a necessary adjunct of the wheel, but is a vitally important one. This fact is not disputed. The question is not whether the velocity head "within" the draft-tube should be directly charged to the wheel, but whether the velocity head at the exit from the draft-tube should be chargeable to the wheel. There is a great difference between the velocity head at the top of the draft-tube, which in low-head installations may represent 50% of the total head acting on the turbine, and the velocity head at the exit from the draft-tube, which may represent only one-quarter of 1% of the total head acting on the turbine.

In the extracts from the author's letters it is claimed that it is impossible to compare properly the efficiency of two turbines unless the draft-tube is eliminated entirely, and, therefore, the subtraction of $\frac{v_3^2}{2g}$ from the value of h_1 in the equation,

$$h_1 = z_1 + \frac{v_1^2}{2g} + p_1,$$

as suggested by the writer, is only a partial step toward the true result needed for comparison.

To illustrate this point, the author has used for comparison two turbines, one of which is closer to the tail-water than the other,

Mr. Taylor. and he points out that in making a comparison between these two, a discrepancy will exist, due to losses in the draft-tube of the turbine which is at the greater distance above tail-water, which are not present in the draft-tube of the turbine which is nearer to the tail-water, resulting in a higher efficiency for the latter than for the former. These losses are due to air leakage, caused by greater vacuum, and also other losses, including friction, due to the greater length of tube.

It is difficult to discuss these suggestions of the author with any degree of satisfaction, before coming to some agreement as to the relation of the draft-tube to the turbine. As already pointed out, the writer contends that the draft-tube is a part of the turbine, as much as the casing, the guide-vanes, or the runner. Any losses due to friction, eddies, whirls, improper rate of change of section in the casing, guide-vanes, or runner, are certainly charged against the turbine. It is the function of the casing to convey efficiently the water from the penstock to the guide-vanes, with as little loss of energy as possible. It is the function of the guide-vanes to transmit this energy as efficiently as possible to the runner, and of the runner to convert this energy into mechanical energy on the shaft. It is the function of the draft-tube to regain the energy at the outflow from the runner as efficiently as possible. If the draft-tube is incorrectly designed, there will be losses within the tube in the form of eddies, whirls, caused by improper change of section, etc., just as there may be losses from these same causes in the casing, guide-vanes, or runner. The writer fails to see just why the draft-tube should be considered in a different light from the casing, guide-vanes, or runner, or why there should be any doubt as to all losses absorbed by the draft-tube being considered in the same light as similar losses in the casing, guide-vanes, or runner.

The turbine builder should be responsible for the design of the draft-tube, and the efficiency which he is enabled to secure depends largely on the type which he uses. If the draft-tube is properly designed and there are, consequently, few losses in this part of the turbine, then the turbine should be credited with a high efficiency, due to an absence of these losses. On the other hand, if the turbine builder, through ignorance, inexperience, or other cause, permits the installation of a draft-tube with improper rate of diffusion, etc., the turbine should be charged with the resulting losses. The outflow loss from the end of the draft-tube, however, as has been stated before, is an entirely different matter. The energy represented is not, nor can it be, effective in the turbine, and therefore it should not be charged against the turbine.

The value of the losses in the draft-tube, and also of the vacuum, is, in the author's opinion, evidently dependent on the vertical distance from the turbine to tail-water. This is not true. The vacuum

and losses within the draft-tube and the length of the tube are not dependent solely on the height of the runner above tail-water, for the question of velocity heads is equally important in establishing the value of the vacuum and the length of the tube. Mr. Taylor.

The author is evidently laboring under the impression that if a turbine is situated near tail-water, the draft-tube will be made only sufficiently long to provide a proper seal below the tail-water level; the draft-tube being, therefore, simply a device for connecting the turbine with the tail-race. It may be pointed out, however, that this use of the draft-tube may be its least important function.

Aside from the question of connecting the runner to the tail-race, the design of the draft-tube is controlled by the necessity of diffusing the velocity of flow from the runner vanes at such a rate as to regain the energy represented therein efficiently.

In high-head turbines, in order to avoid corrosion, and for mechanical reasons, runners of low-speed characteristics are used. In turbines of very low head, in order to secure the highest possible speed of generator, runners of the highest specific speed are used.

With an increase in specific speed, the velocity head at the discharge from the runner buckets approaches a higher percentage of the total head acting on the turbine. Consequently, in a low-head turbine, the value of v_2 may be higher than in a high-head turbine. Therefore, the maximum vacuum in a draft-tube in a low-head turbine may be equal to or very much greater than that in a high-head turbine.

In order that the sum of the static head (the distance from the runner to tail-water level) plus the velocity head from the runner buckets shall not exceed the barometric column, it is necessary in the case of very high specific-speed runners, even under low heads, to place them very close to tail-water level. As a matter of fact, with the extreme values of specific speed used for runners for very low heads, in order that the vacuum within the tube shall not exceed the barometric column, it may be necessary to place the runner below tail-water level. In this case, also, the draft-tube is just as important as, and may be more important than, when the runner is placed considerably above low tail-water level, as the head represented by the velocity at the discharge from the buckets may be, for example, a very large percentage of the total head acting on the turbine, and, in order to secure a high efficiency in the turbine, it is necessary to provide a proper design of draft-tube, which tube would be placed entirely below low tail-water level.

In view of the foregoing, it may be said that it is possible that a turbine with a runner placed very close to tail-water level may have within the draft-tube a higher vacuum than a turbine placed a considerable distance above tail-water; also, the draft-tube in the former case may be considerably longer than in the latter case, as the length

Mr. Taylor. of the draft-tube is governed by the diffusion of the velocity involved, as much as in connecting the runner to the tail-race level. Therefore, the writer does not agree with the author's statement that the tendency toward leakage or the losses within the draft-tube depend on the elevation of the wheel above tail-water.

It follows, therefore, that, as friction and possible leakage losses are not determined by the vertical distance of the runner above tail-water, there is no error introduced by measuring the draft-tube as the writer advocates; namely, by deducting from the vertical distance from the runner to tail-water the velocity head at the outflow from the draft-tube. Therefore, in making a comparative test on two turbines, in which the losses in the draft head are widely different, the efficiency obtained by measuring the draft-tubes, as just described, or as recommended by the writer, will result in not only a true comparison, but in true absolute results respecting the actual efficiencies of the turbines.

Consider, again, the effect of specific speed on the draft-tube: As already pointed out, it is possible that the draft-tube designed to suit a low specific-speed, high-head turbine may be the same as for a high specific-speed, low-head turbine, and that the velocities entering the top of the draft-tube would be the same in both cases. It follows, necessarily, that the percentage of loss due to friction, and other losses in the draft-tube, would be proportionally greater in the case of the low-head turbine, than in the high-head turbine. Hence, the friction loss in the draft-tube is an inherent characteristic of the turbine under consideration, and depends on its specific speed. In view of the fact that there is such an intimate relation between the draft-tube design and the runner, especially in high specific-speed turbines, it would not be logical to consider one as being part of the turbine without the other.

There are so many different ideas among engineers regarding the function of the draft-tube, and the actual performance of the turbine depends so much on the design of the draft-tube, that it is only proper and just that it should be considered a part of the turbine.

The writer will endeavor to point out that, for practical reasons, it is impossible to eliminate the draft-tube, as suggested by the author, even if it were desirable to do so. In order to eliminate the draft-tube, it is necessary to secure the reading, p_2, or the pressure at the top of the draft-tube. It may be pointed out, however, that static piezometer connections to a vacuum gauge on the draft-tube are always subject to erroneous readings. Water at the discharge from the turbine runner is at all gates in a state of whirl about the turbine axis, which whirl may be either in the forward or backward direction, or a combination of both. Whether the piezometer connection is obtained from the wall of the draft-tube, or from a tube inserted into the stream, it is practically impossible to obtain reliable indications of the static pressure or

vacuum actually existing. If piezometer connections are made in the wall of the draft-tube, the centrifugal force resulting from the whirl of the water, in either a well-designed or poorly-designed tube, will always tend to raise the pressure at this point. Thus the apparent vacuum available for the turbine will be less than the vacuum actually existing, and the turbine will be apparently operating under a lower head than is actually the case. Mr. Taylor.

It is safe to predict that, in general, piezometer readings on the side of a draft-tube will not read within 5% of the actual pressure within the tube. This, therefore, illustrates the discrepancy between the values of the head, as given on page 1272, and accounts for the low reading of 137.9 ft., the true value being reduced considerably by the false reading of the piezometer. In view of the fact that false readings result from this whirl, and that the amount of whirl may be entirely different in any two turbines, it follows that the method of eliminating the draft-tube, as suggested by the author, would not give even an approximately fair comparison.

Consider, also, another case, where the method of comparing turbines by the elimination of the draft head would give erroneous results; namely, when the turbine is at too great a height above tail-water. This effect can be shown more clearly by referring to Figs. 13 and 14, which have been reproduced from the author's letter of November 9th, 1914.

Let p_{at} = the height of the barometric column of water;

Let p_{vt} = the vapor tension of the water; in other words, the absolute pressure at which water, at the temperature of that passing through the wheel, will boil or vaporize;

Let L = the friction, eddy losses, etc., in the draft-tube.

Then the greatest possible value of p_2, is

$$p_2 = p_{at} - p_{vt}.$$

In this case p_2 is the actual pressure within the draft-tube, and not the apparent pressure as determined by a piezometer. Therefore, if the turbine is placed above tail-water so that

$$z_2 + \frac{v_2^2}{2\,g} - \frac{v_3^2}{2\,g} - L$$

is greater than

$$p_{at} - p_{vt}$$

the effective head on the turbine will be reduced by the difference. The turbine builder usually decides on the elevation of the unit above tail-water and, therefore, should be charged with this loss of head. The method of measuring the draft head by a piezometer would not in this case indicate the existence of this excessive draft head, and the turbine

Mr. Taylor. builder would not be charged with this head which is wasted; though, as a matter of fact, he should be charged with this loss. This same remark, of course, would apply to a comparison of turbines. Should the draft head of one of these turbines be just up to the limit, and that of the other excessive, the resulting comparison, of course, would be correspondingly erroneous.

It is somewhat surprising to find that a great number of turbines, installed previous to three or four years ago, have excessive draft heads. Even in some of the more modern plants, this condition is found, and, if the head should be measured in the manner proposed by the author, this loss in head due to the excess draft head would not be discovered, and the turbine would get credit for very high efficiency when, as a matter of fact, the high efficiency would not exist.

The writer finds* that the draft head on the 17 000-h.p. turbines at the Tallulah Falls plant of the Georgia Railway and Power Company (head 580 ft.) at full load is calculated to be at least 39.5 ft. The difference between this value and the barometric column, for the particular elevation at which this plant is located, which it is known could not exceed 34 ft., would mean a loss of at least 5½ ft., or practically 1% of the total head on the turbine.

It is very evident that the author's suggested method for the elimination of the draft-tube could not be applied to this case without showing an efficiency in the turbine which does not exist.

In the foregoing the writer has endeavored to point out that it is correct and proper to credit the turbine with the energy represented by the velocity from the end of the draft-tube; and, to meet the possible objection that the turbine builder might restrict the discharge diameter of the draft-tube purposely and unnecessarily, it would be easy to insert a clause in contracts to the effect that the discharge velocity head is not to amount to more than a certain percentage of the head, say one-quarter or three-quarters of 1%, as the case may require. The efficiency of the Cornell turbine is higher by 0.2% than the maximum given by the author, for the turbine has been charged with the energy represented by the exit velocity from the draft-tube, which energy it did not absorb.

Mr. Mead. DANIEL W. MEAD,† M. AM. SOC. C. E. (by letter).—The author has introduced a new method, new, at least, to the writer, for an approximate analysis of turbine characteristics from a very limited number of tests, which is, the writer believes, of considerable value. It must be recognized, of course, that this method is approximate, for the three experimentally determined points through which the author has drawn the torque-speed curves (Fig. 10), could lie equally well on several other curves which could be drawn through these points. The accuracy

* From information in the *Engineering Record*, March 28th, 1914.

† Madison, Wis.

of the results will depend largely on the experience of the investigator. The extended analysis from the limited data of these tests well illustrates the greater value which could be secured at the Holyoke Testing Laboratory, if that Laboratory would uniformly add to its regular tests the determination for each gate-opening tested: first, of the torque at that gate-opening with the wheel at rest; and, second, the run-away speed at such gate-opening with the brake removed. Mr. Mead.

Many of the Holyoke tests are confined to a few experiments near the speed of maximum efficiency, and these experiments are frequently so limited that no deductions can be safely drawn as to what may be expected of the turbine tested if it should be operated at a speed somewhat different from the most efficient one. The value of the tests, both to the manufacturer and to the engineer, would be greatly enhanced, in the writer's opinion, if tests for torque at no speed and for run-away speed were added and, if necessary, fewer tests were made under the conditions of speed approximating those of best efficiency. It is obvious, of course, that the more test points actually determined, the better; but, if any are to be omitted, they should not be in general those at the extreme conditions of speed, for such tests are the limits which largely define the characteristics of the turbine.

R. C. CARPENTER,* M. AM. SOC. C. E. (by letter).—The writer has been much interested in the discussion respecting the method of computing the turbine head which should serve as a basis of efficiency calculations. In Paragraph 6, the author points out the elements of the turbine head, Mr. Carpenter

$$h = z_1 + p_1 + \frac{v_1^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots (A)$$

from which the efficiencies given in the paper were calculated. Mr. Taylor suggests that it would be fairer to the manufacturer if this were reduced by a correction for the velocity of discharge from the mouth of the draft-tube, in which case the head would be represented by

$$h = z_1 + p_1 + \frac{v_1^2}{2g} - \frac{v_3^2}{2g} \ldots\ldots\ldots\ldots (B)$$

In the writer's opinion, it is preferable to base the calculations for efficiency on the total head, as expressed by Equation (A), for although this gives an efficiency which is less than by the other method, it seems to be generally fairer than the method preferred by Mr. Taylor. It is true that the turbine builder may not be responsible for the discharge losses of the draft-tube, but it is also true that the draft-tube is an essential part of the turbine and is usually designed by the builder of the latter. The form of the draft-tube, as well as the construction of the turbine runner, affects this discharge loss, and for that reason

* Ithaca, N. Y.

Mr. Carpenter. the designer should assume the responsibility for the efficiency of the draft-tube. From the standpoint of the purchaser, it is generally desirable that the efficiency which is stated in any proposal, should represent as nearly as possible the total efficiency of the operating plant. In the case of high heads, the effect on the efficiency of using either method of calculating the head is extremely slight.

Regarding the capacity of pumping engines, similar discussions have arisen, although in that particular case the points at issue related to methods of measuring the discharge water by piston displacement as compared with actual measurement. Naturally, the manufacturers have preferred the computations of the discharge by piston displacement, one result of which was to give slightly higher efficiency than the measurement by the actual volume discharged. On account of the difficulties involved in measurements, the piston-displacement method has been used regularly for capacity in the measurement of the pump discharge, and is usually so specified in contracts. The writer does not believe that any injury has resulted to the purchasers of pumping engines by contracting on the basis of piston displacement, and he feels that such would be the case if the head, as specified by Mr. Taylor, were used as the basis of calculation in the sale of turbines, provided such practice was general and was thoroughly understood by purchaser and manufacturer.

The characteristic curves (Fig. 12) which the author has worked out from the turbine test are extremely interesting; in the writer's opinion, they give valuable results which are applicable in the study of the performance of turbines generally, and are quite impossible to obtain with the data usually available in a turbine test.

Mr. Daugherty. R. L. DAUGHERTY,* ESQ. (by letter).—The writer agrees with Professor Zowski that the draft-tube is an integral part of the turbine, and that all losses connected therewith should be charged to the turbine; but, in case it is desired to compare two turbines which have had to be equipped with radically different draft-tubes, due to differences in setting, it would be only fair to eliminate any differences due to the draft-tubes alone. In other words, we are then concerned only with the efficiency of the gates and runner, not with the complete unit. The question is, therefore, how far shall we proceed in the elimination of the draft-tube. This point will be considered subsequently.

It is generally admitted that, for ordinary purposes, the head charged against a turbine should include the draft-tube. The difference of opinion between Mr. Taylor and the writer is as to what constitutes this head. The writer contends that the head consumed by the turbine (including the draft-tube) is the total fall minus the

* Ithaca, N. Y.

penstock losses. The exit velocity at the mouth of the draft-tube is regarded as a loss chargeable to the turbine, owing to the failure of the turbine runner and the draft-tube to abstract all the kinetic energy from the water and reduce this velocity to zero. To be sure, it is impossible to reduce this velocity to zero, but it is equally true that an efficiency of 100% is a physical impossibility in any real machine. The ideal turbine would have to abstract all the energy from the water. Mr. Daugherty

It is the writer's opinion that the turbine is responsible for all the energy delivered to it; Mr. Taylor believes that it should be held responsible for the difference between the energy input and the energy output. The latter term is represented by the kinetic energy of discharge. Mr. Taylor contends that the velocity of the water at the mouth of the draft-tube is influenced by conditions of turbine setting over which the designer has no control. This is true; but it is also true that this velocity for a given setting is also a function of the discharge velocity from the runner and the rate of diffusion provided for in the draft-tube, both being matters in the hands of the designer. It seems to the writer, as well as to Professor Zowski, that the designer must make allowances in his guaranties, where the conditions of setting are unfavorable.

In the comparison of two turbines with different lengths of draft-tubes, which Mr. Taylor quotes, the writer was not sufficiently explicit in his language. The comparison was intended to apply to two identical turbines differing only in their settings. If the turbine runners were likewise decidedly different, Mr. Taylor's criticism would be entirely just.

At present the writer is conducting some tests on a single turbine with five different draft-tubes. Four are of the same length, and differ only in the flare. At one extreme is a tube of uniform diameter, and at the other is one having a flare of 12° for each side, or forming a frustum of a cone the vertex angle of which would be 24 degrees. Thus, there will be quite a variation in the value of v_3. The experimental work is not yet completed, and the writer regrets that he is at present unable to give numerical results which could be used to illustrate these points, but the following observations seem pertinent.

For a given quantity of water supplied to the turbine under a given pressure, it would seem as if the power supplied should be the same, whatever draft-tube is used. It might then be presumed that the efficiency of the turbine with these various tubes would be directly proportional to the power developed. Such would be the results obtained if the equation in Paragraph 6 were used, that is:

$$h = z_1 + p_1 + \frac{v_1^2}{2g} \cdots\cdots\cdots\cdots (A)$$

Mr. Daugherty. The method of computation proposed by Mr. Taylor would give the equation,

$$h = z_1 + p_1 + \frac{v_1^2}{2g} - \frac{v_3^2}{2g} \qquad \text{(B)}$$

According to Equation (*B*), the turbine would be charged with a smaller quantity of power when equipped with the straight draft-tube than when fitted with one having considerable flare. Suppose, for the sake of argument, that the power output of the turbine was the same with all the draft-tubes. As the power output is the same for the same quantity of water delivered to the turbine under the same pressure, it would seem as if the efficiency ought to be unchanged. If Equation (*B*) were used, however, it would indicate that the turbine efficiency is better with the straight tube than when using a draft-tube with some flare. Actually, the power output will not be the same with the four different draft-tubes, but will be greater with a moderate flare than with a straight tube. The true gain in efficiency due to the use of a flaring draft-tube, however, will not be given by using Equation (*B*). The full benefit of the diffusing draft-tube will be realized only when the computations are based on Equation (*A*).

If different power outputs are obtained from this same turbine when fitted with various draft-tubes, all other factors being the same, then, by the use of Equation (*A*), we should obtain different values of the efficiency. These differences are due to the draft-tubes, but the efficiency of the runner alone is absolutely unchanged. Any method of computing turbine efficiency solely for the purpose of comparing two turbine runners, independent of the limitations imposed on them by their settings, should be such as to give identical values when applied to the case in hand. It is perfectly clear that, if the efficiency based on Equation (*A*) increases as the flare of the draft-tube increases, the use of Equation (*B*) will tend to reduce these differences. The writer, however, contends that the only true method is to eliminate the draft-tube and use the equation given in Paragraph 5, for the sake of comparison only. Whether or not this is really practicable is another question.

The writer agrees with Mr. Taylor that it is difficult to determine accurately the pressure at the top of the draft-tube, due to the unknown extent of rotation of the water at that point. Thus the head computed by the equation given in Paragraph 5 may be in error. Some recent computations on this, from another standpoint, have convinced the writer that the value cited on page 1272 is too low, as Mr. Taylor suggests. For this reason it may be, and doubtless is, better to use Equation (*B*), in comparing two turbines, than the equation given in Paragraph 5. This, however, is a case of sacrificing something which

is really logical in favor of something else approximately equal to it, which is more readily obtained. This is similar to the circumstances regarding the measurement of the discharge of displacement pumps, to which Professor Carpenter refers. Mr. Daugherty.

The objection which Mr. Taylor raises, in regard to the determination of the head at the top of the draft-tube, applies also to the determination of the velocity at its mouth. In the test of the Cornell turbine it was noticed, at zero speed on the one hand and at run-away speed on the other, that there was a very decided rotation of the water as it left the tube, the rotation being opposite in direction in the two cases. At the normal speed of the wheel, when the discharge from the runner is approximately "radial", this effect is a minimum, but whatever rotation exists at the top of the draft-tube will persist to the end, though slightly diminished in value. Thus the true velocity of discharge, v_3, will not be obtained by dividing the rate of discharge by the cross-sectional area of the mouth of the tube.

Mr. Taylor's observations of the cases where the draft head is excessive are interesting. That defect in the setting would be covered by either Equation (*A*) or (*B*). The efficiency of the runner alone, however, is independent of this factor, and would suffer if Equation (*B*) were applied. Only the use of the equation in Paragraph 5, if the correct numerical value could be determined, would give the true efficiency of the runner, unaffected by the particular setting.

The writer wishes to express his appreciation of the value of Mr. Taylor's discussion, and especially of his interesting presentation of the effect of the specific speed of the turbine on draft-tube conditions.

The writer's conclusion is that Equation (*A*) should be used in the determination of the efficiency of a turbine in a given setting. He believes that he is upheld in this by practice and by Professor Zowski, Professor Carpenter, the Holyoke testing flume, and by Article 207 of the new test code of the American Society of Mechanical Engineers (1914). Where it is intended to compare two turbines which have widely different draft-tube conditions, Equation (*B*) may be used as being more practical than the other. It is true that, using Equation (*B*), the maximum efficiency of the Cornell turbine is 88.2%, as stated by Mr. Taylor.

As Professor Mead points out, several curves could be drawn through the three points determined for each gate-opening and the resulting characteristic curve is thus only approximate. The accuracy of the interpolated results will depend on the skill and experience of the investigator. A familiarity with the shapes of these curves will aid greatly in their proper construction. However, this paper was intended to point out the possibilities of this method of procedure and show how to obtain a complete set of curves with limited data, rather than to give an accurate analysis of this particular turbine.

Mr. Daugherty. The writer has indicated in the paper how a few other points can be obtained, and, with these additional data, there can be but little question about the exact nature of these curves. Only lack of time to make the test prevented the writer from obtaining a few such special points to illustrate the method further.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1332

SUBMERGED PIPE WORK AT PORTLAND, OREGON*

By D. D. Clarke, M. Am. Soc. C. E.

With Discussion by Messrs. F. M. Randlett, Ben S. Morrow, Clemens Herschel, Kenneth Allen, R. C. Kellogg, Thomas H. Wiggin, W. E. Spear, J. P. Hogan, W. W. Brush, L. J. Le Conte, C. D. Ward, W. R. Phillips, and D. D. Clarke.

Synopsis.

In the following pages reference is made to the experience of the Water Department of the City of Portland, Ore., in supplying a portion of the city by means of several flexible-joint pipes laid under the bed of the Willamette River, a navigable stream passing through the central portion of the city.

The present gravity supply for the city is taken from the Bull Run River, a mountain stream within the limits of the Bull Run Reserve, which adjoins the Oregon National Forest, in the Cascade Mountains, and is brought thence to the eastern border of the city through two steel conduits 24 miles in length, one 33 to 42 in. in diameter, built in 1893 and 1894, and one 44 to 52 in. in diameter, completed in 1911.

The supply for the West Side or main business district is brought across the city through a 32-in., cast-iron main, the river being crossed by one 24-in. and one 30-in. flexible-joint steel main.

The object of this paper is to describe the pipes laid for the river section of this West Side conduit, the method of construction, and

* Presented at the Meeting of December 16th, 1914.

the cost, as well as the steps taken subsequently for the repair and reconstruction of the submerged pipes, as originally laid.

The work described covers:

(1) The laying of a 28-in., cast-iron, flexible-joint pipe in 1894.

(2) The laying of a 24-in., steel, flexible-joint pipe in 1898.

(3) The repair of the 28-in., cast-iron, flexible-joint pipe in 1909.

(4) The laying of a 30-in., steel, flexible-joint pipe in 1911.

(5) The work undertaken during 1913 and now just completed, consisting in taking up and relaying at greater depth the 24-in. line originally laid in 1898, and taking up and storing for future use at another point the 28-in. cast-iron pipe laid in 1894.

In further explanation of the paper, the writer desires to say that in making a search of engineering literature a few years ago he was able to find comparatively few references to flexible-joint pipes of the larger sizes, and therefore he has felt under obligation to place his experience within reach of engineers interested in this subject. For these reasons the following notes have been prepared and submitted.

28-Inch Line.—The pipe laid in 1894 (Plate XXV) consisted of 2 006 ft. of 28-in., cast-iron, flexible-joint pipe, $1\frac{1}{2}$ in. thickness of shell, in net laying lengths of 15 ft. 8 in.

The contract price for furnishing this pipe was a lump sum for a certain specified length, which equalled	$31.4275	per lin. ft.
The contract for laying	2.50	" "
Dredging, 18 746 cu. yd. at 40 cents	3.738	" "
Total	$37.6655	per lin. ft.

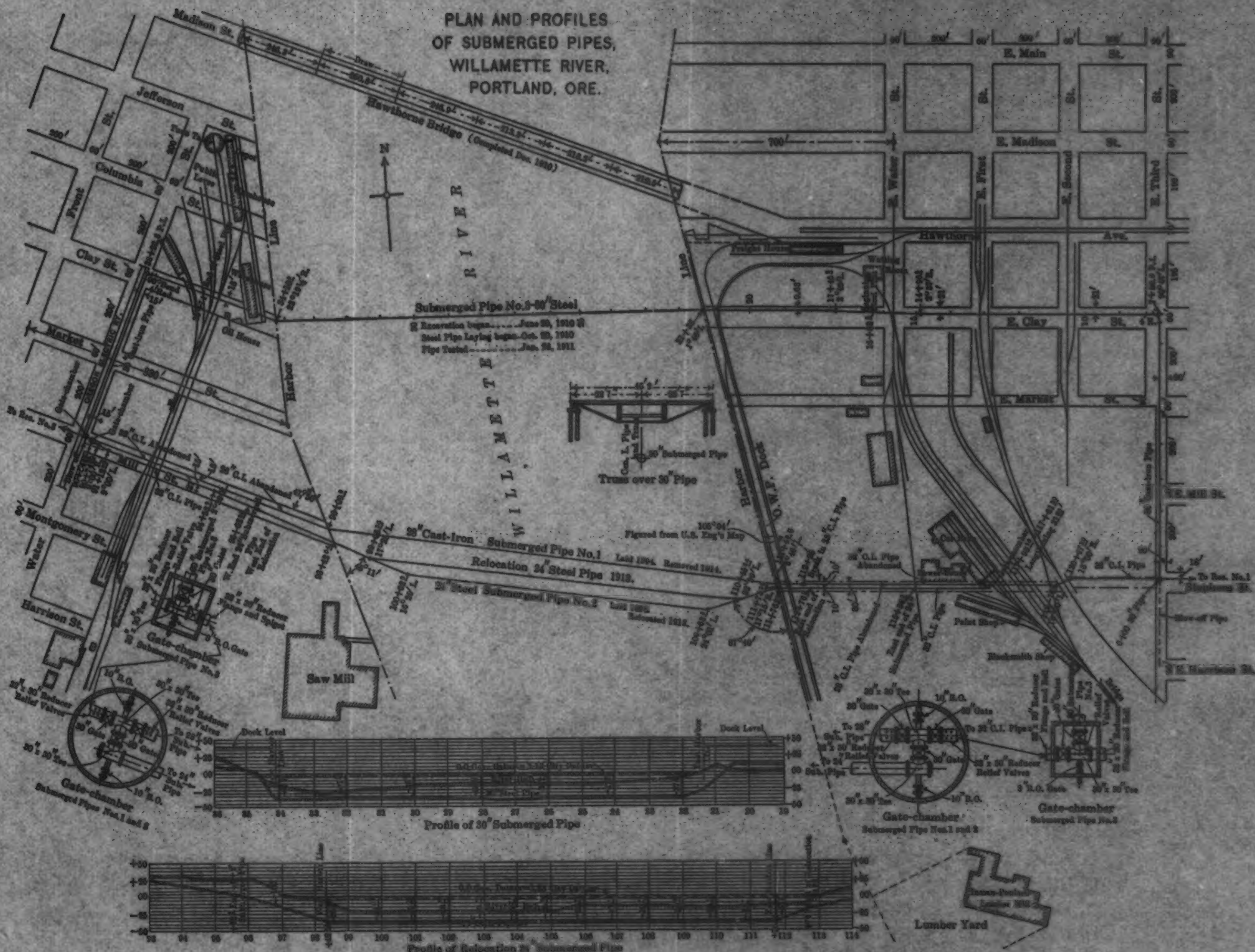
or for 2 006 lin. ft. of 28-in., flexible-joint pipe, in place, a total of $75 556.

During 1894 the Water Department labor wage in Portland was $1.75 per day.

For details regarding this work reference is made to the paper* by Franklin Riffle and the late Albert S. Riffle, Members, Am. Soc. C. E., entitled "A Line of 28-In. Cast-Iron Submerged Pipes Across the

* *Transactions*, Am. Soc. C. E., Vol. XXXIII (1895), p. 257.

PLATE XXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1332.
CLARKE ON
SUBMERGED PIPE WORK.



Willamette River, at Portland, Ore." This paper and the discussions thereon are exceedingly interesting and instructive.

24-Inch Line.—The second pipe, the 24-in. submerged line laid in 1898, consisted of 2 041.6 lin. ft. of lap-welded steel pipe, ⅜-in. plate, including fifty-eight flexible joints, of special design, placed at intervals varying from 20 to 40 ft., depending on the alignment and grade of the pipe trench.

Messrs. Smyth and Howard, of Portland, Ore., were the general contractors for this work, the contract prices being as follows (including two joints and 72 lin. ft. of pipe not used):

60 flexible joints, $260.00 each	$15 600.00
1 938 lin. ft. of 24-in. pipe, $8.50 per lin. ft.	16 472.32
15 332 cu. yd. dredging, $0.40 per cu. yd.	6 132.80
2 041.6 lin. ft. pipe-laying, including joints, $1.77 per lin. ft.	3 613.63
Total	$41 818.75

During 1898 the Water Department labor wage in Portland was $1.75 to $2.00 per day.

This work was undertaken in the late summer, after the subsidence of the June freshet in the Columbia (which sometimes causes a rise of 20 ft. or higher at Portland), and was completed the same season.

The methods and appliances adopted for this work were similar to those adopted in laying the 28-in. line, 4 years before. A ladder-dredge was used in excavating the pipe trench, and the excavated material was taken away on barges to be utilized in filling low ground adjacent to the harbor line in another portion of the city. A cradle was also used in laying the pipe in the excavated trench, as described in the paper by Messrs. Franklin and Albert S. Riffle.

The flexible joint used (Fig. 1) was made in Portland, from designs prepared by the Water Department. Plate XXVI shows the cradle with which the pipe was laid.

The pipe, after it was laid, was tested in a manner similar to that adopted for the 28-in. line in 1894. The results secured did not indicate that the pipe was absolutely tight, but they were accepted at the time as being the best that could be obtained. The final test, under a pressure of from 170 to 180 lb., showed a leakage of approximately 7 gal. per min., or 10 080 gal. per day, or about 8 gal. per lin. ft. of

joint, per day, under the pressures noted, which are approximately 50% in excess of the normal working pressure.

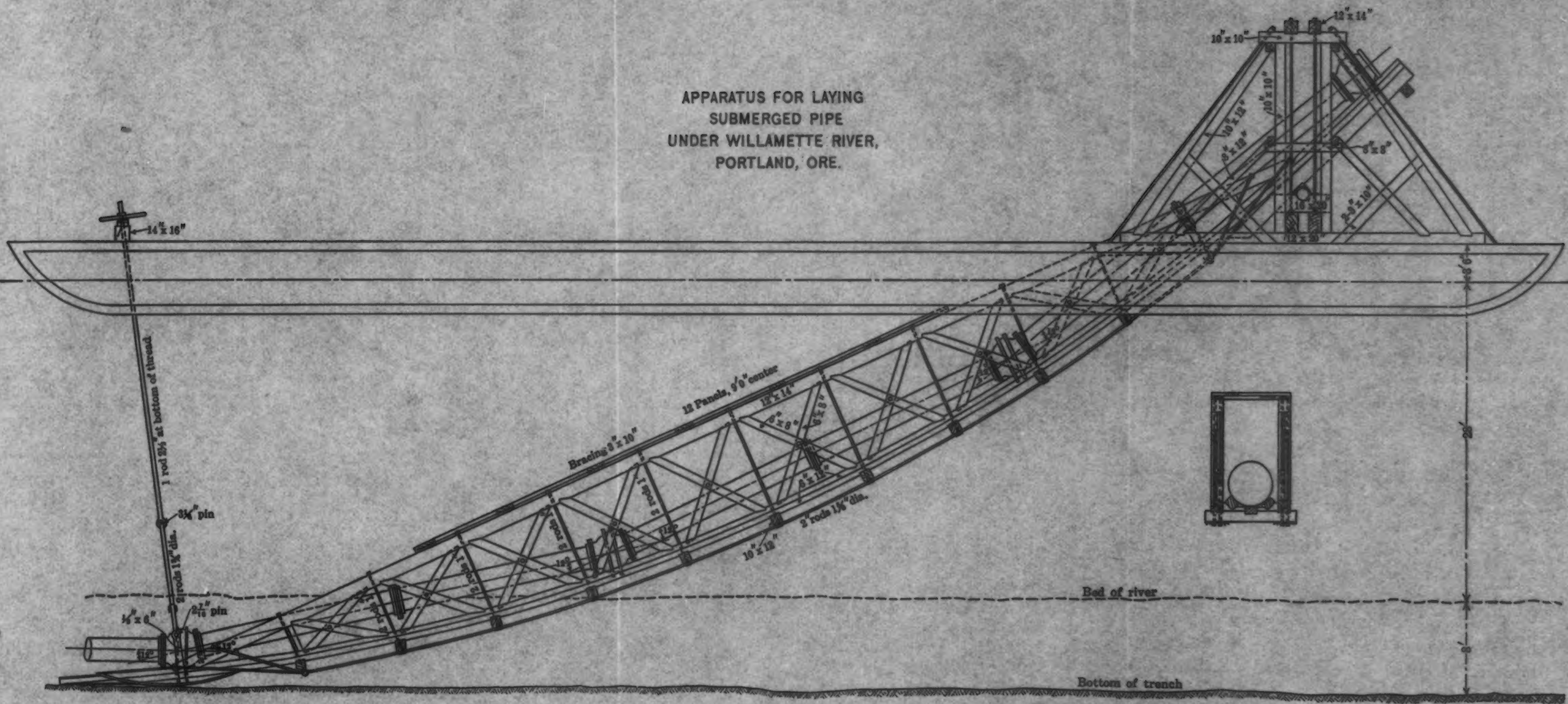
The location selected for this pipe (Fig. 2) was parallel to, and approximately 10 ft. from, the 28-in. line, until the harbor line was reached on each side of the river. The line was then deflected southward, or up stream, until a point was reached 100 ft. from the 28-in. line, which it paralleled for the remaining distance.

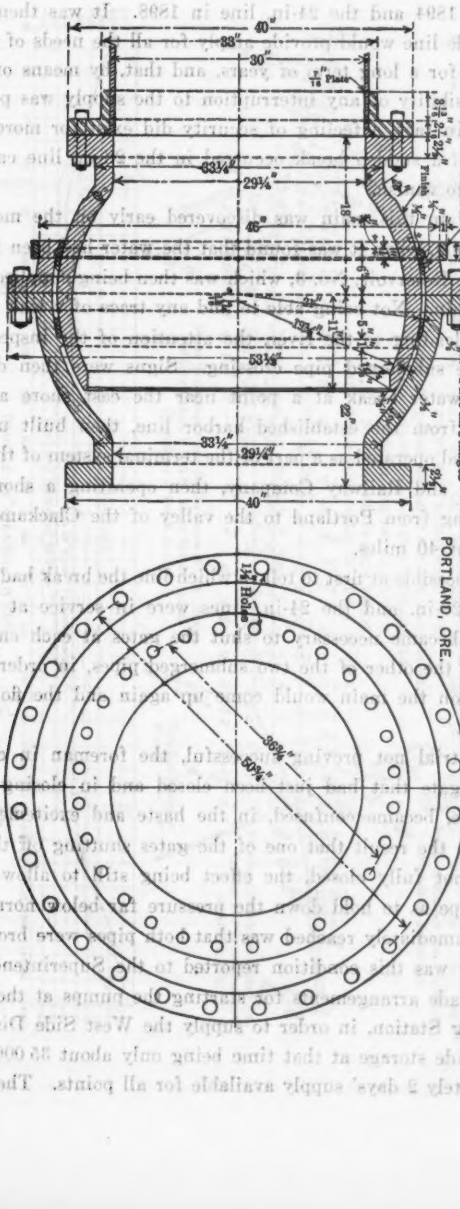
The profile of the original line shows a 22 to 24-ft. channel, 100 ft. or more in width, near the west shore, and, for the larger part of the distance thence to the east harbor line, the low-water depth did not exceed 12 to 16 ft. In establishing the grade for the original line, it was decided first to place the pipe in an open trench dredged to a depth of 8 ft. below the bed of the stream, a cover of 4 ft. being thus provided which was deemed ample to protect the pipe from injury by the passing or anchoring of vessels. In view of the existence of at least a 23 to 24-ft. channel adjacent to the west harbor line, which it was thought would provide for all shipping interests in that portion of the harbor (being at the head of deep-water navigation), it was not thought necessary to incur the extra expense required to dredge a trench to the depth of 30 ft. or more below low water for the entire distance between harbor lines. The matter was compromised, therefore, by placing the pipe grade so as to permit of dredging a channel at least 16 ft. in depth at all points within the harbor limits. This required dredging in places to a depth of from 12 to 14 ft. with at least an 8-ft. trench at all points. The original pipe line (the 28-in.) was laid on a grade established as just described, and the second pipe (the 24-in.) also conformed to the same grade, as nearly as practicable.

For each of the pipe lines heretofore mentioned the flexible-joint portion was a little more than 2 000 ft. in length (2 006 ft. for the 28-in. and 2 041.6 ft. for the 24-in. line). This length covered the distance from river bank to river bank and reaching a point where the grade of the pipe trench was above low-water level in the river. For several hundred feet additional on each side of the river, standard cast-iron pipe was laid to a connection with a concrete gate-chamber, where the two pipes were united with one 32-in. line forming a section of the main Bull Run Conduit for the supply of the West Portland District, the total distance between the east and west gate-chambers being 2 706 ft., and the approximate distance between harbor lines being 1 335 ft.

PLATE XXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1332.
CLARKE ON
SUBMERGED PIPE WORK.

APPARATUS FOR LAYING
SUBMERGED PIPE
UNDER WILLAMETTE RIVER,
PORTLAND, ORE.





BALL AND SOCKET JOINT FOR SUBMERGED PIPE No.3. PORTLAND, ORE.

FIG. 1.

Repair of 28-Inch Line.—As has been stated, the 28-in. line was completed in 1894 and the 24-in. line in 1898. It was then thought that the double line would provide amply for all the needs of the West Side District for a long term of years, and that, by means of the two lines, the possibility of any interruption to the supply was practically eliminated. In fact, a feeling of security did exist for more than 10 years, or until a sudden break occurred in the 28-in. line causing all such dreams to vanish.

The break in this main was discovered early on the morning of April 10th, 1909, when it was found that the water had been falling in the City Park Reservoir, No. 3, which was then being supplied through the broken main. Not being able to find any trace of a leak along the street mains leading to the river, the attention of the inspectors was turned to the submerged pipe crossing. Signs were then discovered of an under-water break at a point near the east shore and about 100 ft. back from the established harbor line, then built up with a dock owned and operated as a part of the terminal system of the Oregon Water Power and Railway Company, then operating a short line of railway leading from Portland to the valley of the Clackamas, a distance of about 40 miles.

It was impossible at first to tell in which line the break had occurred, as both the 28-in. and the 24-in. lines were in service at the time. Therefore it became necessary to shut the gates at each end of first one and then the other of the two submerged pipes, in order to see if the pressure on the main would come up again and the flow become re-established.

The first trial not proving successful, the foreman in charge, in opening the gate that had just been closed and in closing those on the other line, became confused, in the haste and excitement of the moment, with the result that one of the gates shutting off the broken section was not fully closed, the effect being still to allow so much water to escape as to hold down the pressure far below normal. The conclusion immediately reached was that both pipes were broken.

No sooner was this condition reported to the Superintendent than he at once made arrangements for starting the pumps at the Palatine Hill Pumping Station, in order to supply the West Side District, the entire west side storage at that time being only about 35 000 000 gal., or approximately 2 days' supply available for all points. The Palatine

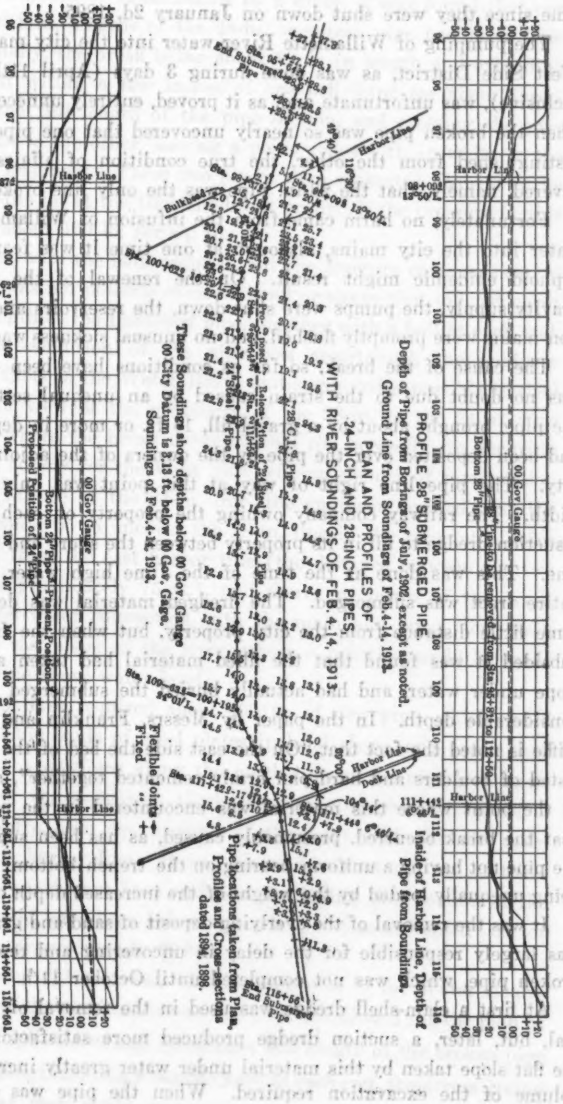

PLAN AND PROFILES OF
24-INCH AND 28-INCH PIPES
WITH RIVER SOUNDINGS OF FEB. 4-14, 1913.
PROFILE 28" SUBMERGED PIPE.
Depth of Pipe from Borings of July, 1902, except as noted.
Ground Line from Soundings of Feb. 4-14, 1913.
Inside of Harbor Line, Depth of
Pipe from Soundings.
These Soundings show depths below 00 Gov. Gauge
00 City Datum is 8.13 ft. below 00 Gov. Gage.
Soundings of Feb. 4-14, 1913.
Pipe locations taken from Plan,
Profiles and Cross-sections
dated 1894, 1898.
PROFILE 24" SUBMERGED PIPE
Depth of Pipe from Soundings of Dec. 29, 1898.
Ground Line from Soundings of Feb. 4-14, 1913.
Harbor Line
Dock Line
Bulkhead
00 Gov. Gauge
Flexible Joints
Fixed
End Submerged Pipe

FIG. 2

pumps were started at about 10.00 A. M. on April 11th, for the first time since they were shut down on January 2d, 1895.

The pumping of Willamette River water into the city mains in the West Side District, as was done during 3 days (April 11th to 13th, inclusive), was unfortunate and, as it proved, entirely unnecessary, for, when the broken pipe was so nearly uncovered that one pipe could be distinguished from the other, the true condition of affairs was discovered, namely, that the 28-in. pipe was the only line broken.

Fortunately, no harm came from the infusion of Willamette River water into the city mains, although at one time it was feared that a typhoid epidemic might result. On the renewal of the Bull Run gravity supply, the pumps were shut down, the reservoirs and distribution mains were promptly flushed, and no unusual sickness was reported.

The cause of the break, so far as conditions have been diagnosed, was no doubt due to the strain caused by an unequal settlement of the pipe, brought about by a gravel fill, 16 ft. or more in depth, which had been deposited over the pipe by the owners of the adjoining property. The pipe line right of way at this point was only 30 ft. in width. The railway company owning the property on each side used a suction dredge to fill in its property between the shore and the harbor line. This was done at the time of the June high water, when the entire tract was submerged. The dredged material was deposited at some little distance from the city property, but when the freshet had subsided it was found that the filled material had taken a very flat slope under water, and had actually buried the submerged pipes to a considerable depth. In the paper by Messrs. Franklin and Albert S. Riffle is noted the fact that "On the east side the bed of the river consisted of boulders and hard pan firmly cemented together", and it was at the point where this material was encountered in the pipe trench that the break occurred, presumably caused, as has been suggested, by the pipe not having a uniform bearing on the trench bottom, and hence being unequally loaded by the weight of the increased depth of fill.

It was the removal of the overlying deposit of sand and gravel which was largely responsible for the delay in uncovering and repairing the broken pipe, which was not completed until October 11th.

At first a clam-shell dredge was used in the removal of this material, but, later, a suction dredge produced more satisfactory results, the flat slope taken by this material under water greatly increasing the volume of the excavation required. When the pipe was uncovered

sufficiently to admit of examination by a diver, it was found that one of the 15 ft. 8-in. lengths of the 28-in., cast-iron pipe had been fractured diagonally, the break being about 12 ft. in length and extending from a point within a few inches of one bell joint for about three-fourths of the full length of the pipe section, and crossing the pipe from one side to the other, leaving a point about 1 ft. in width at the end of each fragment. These two pieces were separated about 3 ft. by the force of the jet at the time the break occurred, the pressure at this point being about 150 lb. per sq. in.

In no case were the balls injured, but the fracture came so near to them that it was not possible to use a band or sleeve that did not cover the entire length of the fractured pipe, together with a ball at each end.

When the condition of the pipe was ascertained, the engineer reported the facts to the Water Board and suggested a possible method of repair. In view of the proximity of the 28-in. to the 24-in. pipe at the point where the break occurred, and the necessity for securing a continuous supply for the West Side without danger of interruption through the failure of the 24-in. line, the engineer decided that the safest plan would be to uncover the 28-in. line in the river at a point about 100 ft. from the east harbor line (the distance between the two pipes being approximately 75 ft.), and there, by means of the diver, make an under-water connection with the main and lay a new main thence to the east shore of the river, where a junction could be made with the old line above water level.

This would have required about 400 ft. of new pipe, which could have been quickly made of steel plates in local shops, and could have been laid from a platform along the north line of the pipe line tract. The only work in connection therewith requiring the services of a diver would have been the making of the joint with the old main in front of the harbor line. It was planned so that the use of three or four joints would have been sufficient for laying that portion of the pipe which would have been below water level. The estimated cost of this work was approximately $9 500.

The foregoing did not meet with the approval of the Water Board, owing largely to the cost, and it was therefore abandoned for the plan of constructing a split sleeve and encasing a full length of the broken pipe in its original bed, including a flexible joint at each end, the divers giving assurances that they could make the necessary joints under water.

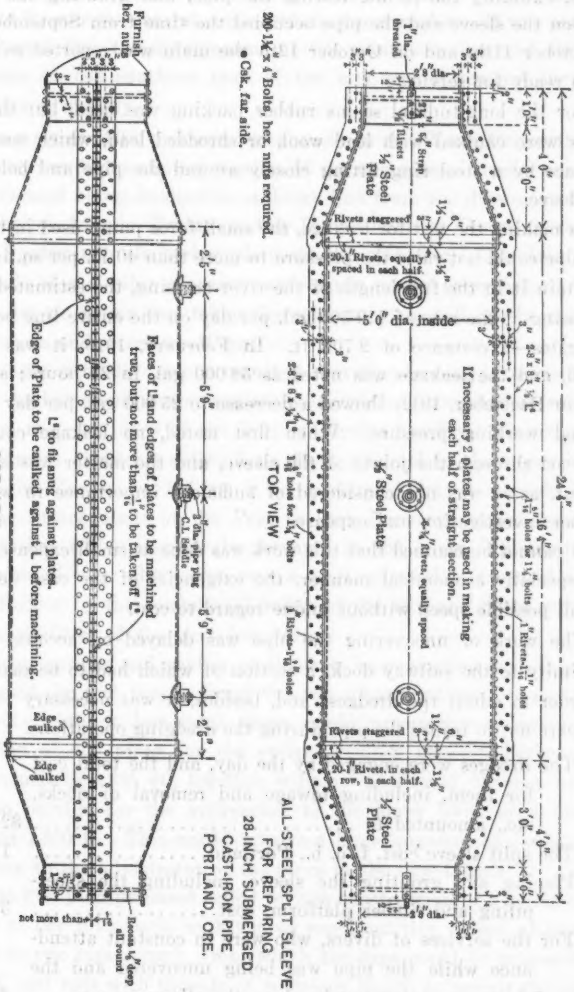
Owing to the fact that at the point where the break occurred the two pipes were near to one another (approximately only 2 ft. between them for a short distance) and that the broken pipe was also about 2 ft. lower than the 24-in. steel pipe which must be kept in service, it was determined that, prior to undertaking the repair work, and in order to protect the 24-in. main thoroughly, a row of timber sheet-piling should be driven between the two pipes, so as to avoid any danger of undermining the steel main by the excavation for the broken pipe. The pressure on this main being approximately 150 lb. at the bed of the river, it was decided to reduce this considerably before the piles were driven. This was done by shutting down the flow during the day, when the work of pile-driving was in progress, and turning it on again at night in order to maintain the supply in the City Park Reservoirs. One hundred and ten 9 by 12-in. "Wakefield" piles were driven, which occupied 5 days, and on the completion of this work, the additional dredging required to expose the 28-in. line fully was undertaken.

Preparatory to placing the split sleeve designed for this purpose (Fig. 3), the ends of the broken pipe were lifted partly from the position in which they were found, and also moved horizontally a short distance from the sheet-piling in order to give room for the sleeve. The fractured ends of the pipe were drawn together as closely as possible and then wired in place and afterward wrapped in heavy tarred canvas, securely fastened, the object of the latter being to prevent the cement grouting placed inside the sleeve from gaining admission to the interior of the pipe.

The sleeve was designed to be built of riveted steel plates, of sufficient diameter and length to take in an entire length of the pipe, with a flexible joint at each end. This meant that the main barrel of the sleeve must be about 4 ft. in diameter and 16 ft. long, with a reducer at each end, diminishing from 4 ft. to a diameter of about 32 in. at each end in a distance of about 3 ft., thus fitting the outside of the 28-in. pipe, making the extreme length over all 22 ft.

A contract for the construction of this sleeve was awarded to the Moran Company for $1 026.50, and it was placed by Department forces. The weight of the sleeve was 14 100 lb.

The progress of the work was delayed by the high stage of water due to the June flood in the Columbia River, and time was also required to secure the necessary materials and manufacture the sleeve. The sleeve was not received until September 8th, but the placing of it



ALL-STEEL SPLIT SLEEVE FOR REPAIRING 28-INCH SUBMERGED CAST-IRON PIPE PORTLAND, ORE.

FIG. 3.

was commenced immediately thereafter. The work of placing the sleeve, caulking the joints, testing the pipe, and grouting the space between the sleeve and the pipe occupied the time from September 8th to October 11th, and on October 12th the main was reported as being again ready for service.

For the longitudinal seams rubber packing was used, but the end joints were caulked with lead wool, or shredded lead, which was held in place by a steel ring fitting closely around the pipe and bolted to the sleeve.

In making the test for leakage, the small force pump used in testing the pipe could not raise the pressure to more than 40 lb. per sq. in. and maintain it in the full length of the river crossing, the estimated leakage being at the rate of 119 500 gal. per day on the entire line between the gates—a distance of 2 706 ft. In February, 1910, it was again tested, and the leakage was noted as 38 000 gal. in 24 hours; a third test, in December, 1911, showed a decrease to 25 000 gal. per day under normal working pressure. When first tested, no leakage could be observed through the joints of the sleeve, and the matter was allowed to rest, as it was not considered of sufficient importance to warrant further investigation and expense.

It cannot be claimed that this work was done at small expense, or in an especially economical manner, the exigencies of the case demanding all possible speed without undue regard to cost.

The work of uncovering the pipe was delayed on account of its proximity to the railway dock, a section of which had to be taken out in order to admit the dredges; and, besides, it was necessary to exercise care not to injure the pipe during the dredging operations.

The dredges were engaged by the day, and the total cost for them, including towage and removal of docks, etc., amounted to	$2 850.00
The split sleeve cost, f. o. b., Portland	1 026.50
Placing and grouting the sleeve, including the sheet-piling and timber platform, cost	3 138.00
For the services of divers, who were in constant attendance while the pipe was being uncovered and the sleeve was being put in place, the allowance was	2 250.00
The total cost of the repairs, including the items just mentioned and incidental expenses, was	$10 750.00

Lowering the Pipes.—As already stated, the grade for the first pipes laid was established so as to provide for an available channel depth of only 16 ft. over the central portion of the upper harbor. It was not then anticipated that the development of the manufacturing and shipping interests in the southern part of the city would demand increased facilities so quickly.

As early as 1902, a demand was made on the Water Committee for the lowering of the central portion of the two pipes, in order to admit of a channel being dredged on a direct line from the draw-span of the Hawthorne Avenue Bridge to the Inman-Poulsen Company mill—a large lumber manufacturing plant on the east side. Complaints and demands of a similar character were made nearly every season thereafter, but no action was taken thereon, except to call on the engineer for reports and estimates of cost.

In 1911 it finally became manifest that the shipping interests in the upper harbor would soon require improved facilities, and it was also learned that the plans of the Port of Portland and the United States Government engineers called for a 30-ft. channel in the upper harbor above the Hawthorne Bridge, it being claimed that this would suffice for the future needs of the Port in that direction, regardless of the depths that will ultimately be required in the lower Willamette and Columbia Rivers, where even now a 40-ft. channel is dreamed of and talked about by the commercial bodies of the city and State as among the possibilities of the next few years.

Investigations and studies undertaken in connection with this work developed the fact that to lower the two submerged pipes so as to give a 30-ft. depth at low-water stage would practically necessitate the lowering of the pipes for the entire distance between harbor lines, approximately 1 335 ft., together with an additional distance at each end to provide for the approaches to the lower level, or a total of at least 1 500 ft. This would involve a large quantity of dredging, in addition to the adjustment of the pipes in the new trench.

The first plan prepared called for the construction of a series of pile bents of two piles each, with timber caps and ties, placed at intervals equal to one pipe length, from which the pipe could be suspended by slings, and rods with threads of sufficient length to permit the pipe to be lowered into the new position after the supporting earth had been removed and the new trench excavated. It was thought that the

necessary excavation could be made from the side of the pipe with a suction dredge which could be manipulated so as to undermine the pipe gradually and admit of lowering it.

This plan was abandoned later, owing, in part, to the difficulty in handling the dredged material which had to be barged away and disposed of outside of the river channel.

The method finally adopted called for taking up the two pipes and, after making necessary repairs, relaying them in the trench excavated to the required level, as described later.

30-Inch Line.—After considering the difficulties attending the lowering of the two pipes and the danger of breaking one or both of them, and the consequent interruption of the supply for the entire West Side, or main business district of the city, the Water Board decided that, in the interests of safety, a new and entirely separate submerged pipe line should be laid in advance of any work which might in any manner disturb the two pipes then in use. Plans and specifications were therefore prepared by the office staff calling for a 30-in. line crossing the river at the foot of Clay Street, or approximately 600 ft. distant from the two pipes already in use.

The new line was planned to leave the 32-in. supply main at a point about 400 ft. east of the original East Side gate-chamber at 3d and Stephens Streets, and to connect again with the 32-in. main at the West Side gate-chamber at Water and Mill Streets.

This plan called for 4 312.6 ft. of entirely new pipe. Of this, 2 265.6 ft. on the east and west sides were to be standard, 30-in., cast-iron pipe, Class "F", New England Water Works Association specifications, and the remaining 2 047 ft. were to be 30-in., lap-welded, galvanized and asphalted steel pipe, of $\frac{7}{16}$-in. plate with flange joints. Inserted in the steel main, at intervals of about 60 ft., were placed twenty-seven flexible joints of cast iron.

The grade established for the pipe trench between the harbor lines was 38 ft. below low-water level, which provided for a 3 to 4-ft. cover over the top of the pipe, with a 30-ft. depth of channel, as called for by the plans of the Port of Portland and the U. S. Engineers.

This work was commenced in August, 1910, and completed in March, 1911.

The dredging of the trench for this main was commenced by one of the suction dredges operated by the Port of Portland Commission,

at a rental of $250 per day. The work with this dredge was soon discontinued, however, as it was needed for channel work at another point, and the dredging was completed with the dipper-dredge owned by the Pacific Bridge Company, this Company being in a position to utilize the dredged material for filling purposes in a near-by district.

The 30-in., standard, cast-iron pipe used for the shore connections was manufactured by the Oregon Iron and Steel Company, at its foundry near Portland, the price, in place, being $11.10 per ft.

The flexible joints were made by the John Wood Iron Works, of Portland, from plans furnished by the Water Department, for $345 each.

The 30-in., lap-welded steel, galvanized and asphalted pipe was purchased from the National Tube Company (2 200 ft.) at $14 per ft.

The contract for laying the pipe and making the joints was awarded to Robert Wakefield and Company, of Portland, for the prices given in Table 1.

TABLE 1.—COST OF PIPE LAYING.

Trench excavation	5 705 cu. yd.	at $0.90	$5 134.50
Cast-iron pipe in place	2 265.6 ft.	" 11.10	25 148.16
Steel pipe, with flexible joints, laid	2 047 lin. ft.	" 4.50	9 211.50
Ball-and-socket joints (only 27 used)	30	" 345.00	10 350.00
Port of Portland dredge, estimated cost of	102 655 cu. yd.		16 160.14
Pacific Bridge Company's dredge, estimated cost of	29 700 cu. yd.		4 158.00
Valves, 30-in.	4	at $590.00	2 360.00
Concrete, in two gate-chambers	59 cu. yd.	at 12.00	708.00
Specials	43 962 lb.	at 0.06	2 637.72
Extras: labor and materials, dock repairs, etc.			10 197.84
Steel pipe used	2 047 ft.		28 658.00
Total			$114 723.86

SUMMARY OF TOTAL COSTS:

Steel pipe	$29 135
Dredging	20 318
Cast-iron pipe and pipe laying	65 747
Labor and engineering	4 808
Total	$120 008
Original estimate	125 000

On the completion of the work, the pipes, including the standard cast-iron and flexible-joint steel pipe, were tested to 175 lb. per sq. in., and the leakage under the normal pressure of 150 lb. was found to be approximately 30 000 gal. per day.

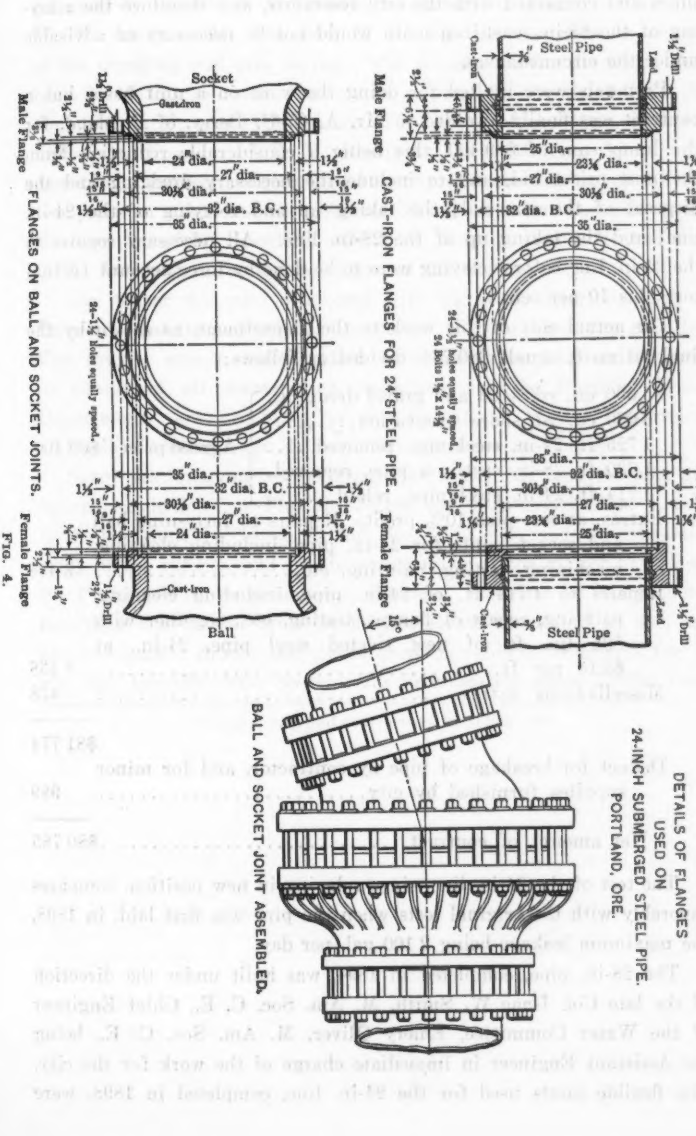
Having become satisfied that this loss was from no one leak of special size, but rather from a number of small leaks, no attempt was made to check them. It may be stated that the last test was made on September 28th, 1912, when the seepage loss was at the rate of 5 000 gal. per day—a decrease of 25 000 gal. per day from that which occurred shortly after the main was laid.

Lowering of 24-Inch Line.—Soon after the completion of the new line, in 1911, a renewed application was made by the milling interests and the Port of Portland Commission, for the removal of the original submerged pipe lines, which were claimed to be a serious obstruction to navigation in the upper harbor.

An attempt was made to induce the Port of Portland Commission to share in the expense of this work, at least to the extent of contributing the use of its dredges for the removal of the material which it would be necessary to excavate to uncover the pipe and prepare the new trench at a lower level, but the inability of the Water Department to secure a satisfactory dumping ground prevented the consummation of the arrangement finally agreed upon, namely, that the Port of Portland would deduct from the sum due for the use of its dredges (at $350 per day) a sum sufficient to cover the cost of dredging a channel 300 ft. wide at the submerged pipe crossing a distance of 100 to 200 ft. on the line which had been planned, extending from the draw-span toward the upper river channel. This comprised a very small part of the total work required for this channel, which had been laid out in such a position as to oblige the city to lower its pipes for the entire distance between harbor lines, the natural deep-water channel near the west harbor line having been entirely ignored.

The conclusion of the whole matter was a decision of the Water Board to advertise for bids for the entire work, independent of any promise or expectation of aid from any source. It was furthermore decided to ask for proposals, on a unit basis, for the necessary dredging and the disposal of the dredged material; for uncovering and taking up both the 28 and 24-in. pipes; and for relaying the 24-in. pipe only—the 28-in. pipe to be stored for future use at some other crossing.

It was deemed that the two steel pipes, 30 and 24 in. in diameter, respectively, would provide an ample supply for the West Side District, the estimated capacity of the two pipes being nearly 40% in excess of that of the 32-in. line forming the shore ends of the submerged



FLANGES ON BALL AND SOCKET JOINTS.

FIG. 4.

CAST-IRON FLANGES FOR 24" STEEL PIPE

DETAILS OF FLANGES USED ON 24-INCH SUBMERGED STEEL PIPE. PORTLAND, ORE.

BALL AND SOCKET JOINT ASSEMBLED.

pipes and connected with the city reservoirs, and therefore the relaying of the 28-in., cast-iron main would not be necessary or advisable under the circumstances.

Proposals were invited for doing the work on a unit basis, but a contract was finally awarded to Mr. A. C. U. Berry, of Portland, for the lump sum of $69 400, this being a considerable reduction from the unit prices bid, and to include the necessary dredging and the disposal of the material; the taking up and relaying of the 24-in. line; and the taking up of the 28-in. line. All necessary repairs to the 24-in. line before relaying were to be done by force account (actual cost plus 10 per cent.).

The actual cost of this work to the Department, as shown by the final estimate, equals $80 785, divided as follows:

<table>
<tr><td>85 000 cu. yd. sand and gravel dredging.
6 060 cu. yd. shore excavation.........
1 725 ft. 24-in. steel pipe removed.....
1 630 ft. 28-in. cast-iron pipe, removed.
1 714 ft. 24-in. steel pipe, relaid.......</td><td>Agreed price: $69 400</td></tr>
<tr><td>Extras, at cost plus 10% profit. Repairs to forty-nine ball and socket joints for 24-in. pipe, including cleaning, machining, testing, painting, etc...................</td><td>3 438</td></tr>
<tr><td>Repairs to 1 714 ft. of 24-in. pipe, including cleaning, painting, repair of flanges, testing, etc., together with 808 lin. ft. of new riveted steel pipe, 24-in., at $5.15 per ft.....................................</td><td>8 458</td></tr>
<tr><td>Miscellaneous extras...........................</td><td>478</td></tr>
<tr><td></td><td>$81 774</td></tr>
<tr><td>Deduct for breakage of pipe by contractor, and for minor supplies furnished by city..........................</td><td>989</td></tr>
<tr><td>Net amount of contract..............................</td><td>$80 785</td></tr>
</table>

The test of the 24-in. line, since relaying in new position, compares favorably with the original tests when the pipe was first laid, in 1898, the maximum leakage being 2 100 gal. per day.

The 28-in. pipe, completed in 1894, was built under the direction of the late Col. Isaac W. Smith, M. Am. Soc. C. E., Chief Engineer of the Water Committee, Emery Oliver, M. Am. Soc. C. E., being the Assistant Engineer in immediate charge of the work for the city. The flexible joints used for the 24-in. line, completed in 1898, were

designed by the late J. A. Lesourd, mechanical engineer, of Portland, W. W. Amburn, M. Am. Soc. C. E., was the Assistant in charge of the dredging and pipe laying. The joints used for the 30-in. line, built in 1911, were made from plans prepared by W. R. Phillips, M. Am. Soc. C. E., also of Portland. The laying of the 30-in. line was under the direction of F. M. Randlett, Assoc. M. Am. Soc. C. E., Assistant Engineer, Water Department; and the taking up of the two pipes in 1913, and the relaying of one of them, was also under the direction of Mr. Randlett, with B. S. Morrow, Assoc. M. Am. Soc. C. E., Assistant in immediate charge of the work.

The writer was also connected with the work described, first as Principal Assistant Engineer in charge of pipe lines for the Bull Run gravity supply, completed in 1894, and since 1897 as Engineer in charge of all construction operations for the Portland Water Department, now organized as the Bureau of Water, Department of Public Utilities, Will H. Daly, Commissioner in Charge.

Since the foregoing was prepared the writer has read the able and instructive article on Flexible Joints for Submerged Pipe Lines,* by the late Emil Kuichling, M. Am. Soc. C. E., who made an exhaustive study of the subject, and by his presentation of the matter has placed the Profession under obligations therefor. The data he has gathered regarding the several types of flexible joints heretofore in use, and his conclusions drawn therefrom, form a valuable reference for all who may hereafter have occasion for investigation along similar lines.

* *Engineering and Contracting*, April 15th, 1914.

DISCUSSION

Mr. Randlett. F. M. RANDLETT,* ASSOC. M. AM. SOC. C. E. (by letter).—A short discussion of the methods and plant used by Robert Wakefield and Company in the construction of the 30-in. submerged pipe line at Portland, Ore., as described by the author, may be of interest. Mr. A. C. U. Berry, who later was the general contractor for removing the 24-in. and 28-in. and relaying the 24-in. submerged pipes, had charge of the work for Robert Wakefield and Company.

The shore ends of the 30-in. bell-and-spigot, cast-iron pipe were laid in the usual manner. When the first tests of this line were made, three leaky joints were repaired and one cracked pipe was replaced. This part of the work was commenced as soon as the contract was signed, and was ready for the steel pipe connections some time before the dredging was completed and all the steel pipe had been delivered.

The steel pipe was furnished to the contractor, f. o. b. cars, by the City, and the contractor handled the pipe from the cars to the work and stored about 40 ft. of extra pieces in the City yard. This pipe was standard, 30-in. outside diameter, lap-welded steel, galvanized and dipped, made of $\frac{7}{16}$-in. plate, and scheduled to weigh 176.848 lb. per ft. The total shipment, as reported by Robert W. Hunt and Company, inspectors for the City, consisted of 2 081 ft., weighing 464 469 lb., including the flanges. In addition, the order provided for 3 500 $4\frac{3}{8}$ by $1\frac{3}{8}$-in. bolts with hexagonal nuts, weighing 17 730 lb.

It was contemplated that the flanges would be riveted to the pipe, but it was decided later, after solicitation from the mill, to allow the pipe to be fitted to the flanges and peened, the flanges being then lathe-faced and the holes accurately spaced and bored.

The ball-and-socket joints, made by the John Wood Iron Works, of Portland, Ore., were rotated and tested before leaving the shop, and then hauled and placed by the contractor. Each joint weighed about 5 700 lb.

The method of placing the submerged portion of the line differed from any previously used, in that no special crib, cradle, or tools were required, other than three derrick barges and two common deck barges. Previous lines across the river had been laid with cradles specially constructed for the purpose. By using the plant as described, the work of laying the submerged portion required only 9 days, thereby probably saving considerable expense and causing no noticeable delay to the river traffic.

The work on this submerged part of the line was commenced on the east side of the river. It was necessary to remove a portion of the O. W. P. Dock in order to lay the pipe on the incline just east of

* Portland, Ore.

the harbor line. After placing the inclined section and two additional lengths west of the bottom of the slope, the procedure was as follows: One derrick held the westerly end above the water, while three, and occasionally four, lengths of pipe with one flexible joint (already bolted together) were bolted to it; the derrick then lowered the pipe until only the westerly end was above the water. The total lift varied from 8 to 12 tons before submerging. Mr. Randlett.

The slings were marked and care was taken while lowering that the flexible joints were not allowed to deflect more than 11 degrees. The rear slings were then slipped, and the operation was repeated. In lowering the pipe it was necessary to permit water nearly to fill the pipe, as it was almost buoyant, and it was desirable to keep the strain on the three derricks as nearly alike as possible.

The dredging was done by the Water Department with a 20-in. hydraulic suction-dredge, owned and operated by the Port of Portland Commission, and the *Titan,* a large dipper-dredge owned by the Pacific Bridge Company. The suction-dredge excavated most of the material, the dipper-dredge being used only when it was necessary for the suction-dredge to be used by the Port of Portland on lower river work.

The only delay was caused by the failure of the steel pipe to arrive when expected, which resulted in an extension of the time set for the completion of the work. All parties concerned in the work co-operated in every possible way, and the various parts progressed without delay. Progress on the West Side, under the Southern Pacific and the Oregon Electric Railway tracks, was slow, but no interruption to traffic occurred.

While placing the pipe in the river, one of the slings broke, damaging one ball-and-socket joint and the ends of two lengths of pipe. The damage was quickly repaired by the contractor, and no delay was caused thereby.

The valves on this line are extra heavy, 30-in., Rensselaer, double-disk, gate-valves, with by-pass and indicator. There are two 4-in., Lunkenheimer, relief valves on each end of the line at the gate-chambers, and set to blow off at 135 lb., which is 15 lb. in excess of the average working pressure at the points mentioned.

The gate-chambers are of concrete, with reinforced concrete top and standard manhole covers. There is a standard cast-iron gate-box casting in the concrete top over the operating nut of each valve, so that the latter may be operated from the street with the regulation gate-key.

A portion of the steel pipe on the East Side of the river lies on a gravel fill between East Water Street and the harbor line, and is not completely covered by gravel. During the high-water stage of the river this pipe is entirely submerged, but is exposed at all other times. A recent examination of this pipe shows absolutely no indica-

Mr. Randlett. tion of pitting or rusting, although the dip coat is practically gone, the galvanizing being exceptionally clear and bright.

Tests of this pipe line in May and June, 1914, showed a leakage of about 1 000 gal. per 24 hours.

Mr. Morrow. BEN S. MORROW,* ASSOC. M. AM. SOC. C. E. (by letter).—A brief description of the method and equipment used in carrying out this work may be of interest.

In connection with taking up and relaying the 24 and 28-in. lines, the dredging work was naturally divided into three operations: (1) Removing the cover over the 24-in. line; (2) excavating a trench for the new location of the 24-in. pipe to a depth of 38 ft. below low water for the entire distance between harbor lines; and (3) removing the fill over the 28-in. pipe.

The work, in the main, was done by a dripper-dredge capable of handling 1 400 cu. yd. of sand and gravel in an 8-hour day (sand and gravel constituting practically all the dredged material). This dredge could work to a depth of 44 ft. below water surface, and the stage of the river was such that during the entire operation it was possible to dig to the required depth of 38 ft. below low water. An effort was made to use a floating derrick operating a clam-shell bucket in uncovering the 24-in. pipe, but it was found that the bucket was damaging the pipe, and its use was discontinued.

The average cost of dredging and removing the material was approximately 12 cents per cu. yd.

The removal of the 24-in. line was begun on the west bank of the river, and the pipe was disconnected above the water level. The first six lengths were taken up by a land derrick. Slings were put around the pipe by a diver, and care was taken not to rotate any flexible joint through a greater angle than 15°, which was the maximum possible movement for the ball-and-socket joint. By raising a pipe a few feet and slinging it to the bents on which the land derrick operated, and then raising the next pipe a few feet and repeating the operation, the pipe was taken up out to the point where the two floating derricks could take hold.

Very early in the course of this work it was found that, when a flexible joint was rotated through a greater angle than the allowable 15°, the joint where the steel pipe was leaded into the cast-iron flange began to give, and under a slight strain the pipe would pull out entirely. This was due to the fact that the lead ring in the flange was not as deep as required by the present standard. On examination it was seen that the pipe pulled on the flange without damage to itself or the ball-and-socket joints. The contractor took advantage

* Portland, Ore.

of this condition, and, putting slings on two or three pipes at a time, brought it up piecemeal, without doing it any material damage. Mr. Morrow.

This pipe was taken up by a crew consisting of a foreman and seven laborers, a diver and helper, and two floating derricks, of 45 and 25 tons capacity, respectively, at a cost of $1.70 per ft. The pipe was taken up to a point 100 ft. inside the east harbor line, where it was disconnected under water by a diver.

When originally laid this pipe had been dipped in a mixture of pure-graded refined California asphalt, fluxed with a high-grade natural liquid asphalt heated to 280° Fahr., and, although it had laid under the river for 16 years, the coating was absolutely intact and, with one exception, the plate was in as good condition as when laid.

After the pipe was taken out, the flanges were removed and the pipe thoroughly sand-blasted. The lead rings on the flanges were made deeper and the pipes were redipped in a mixture of 80% "Manco" pitch and 20% California asphalt, "F" grade, at a temperature of 300° Fahr. This mixture has a melting point of 175°, and is entirely flexible at 32 degrees. It shows no tendency to become soft and sticky at 100° Fahr.

The ball-and-socket joints were taken apart and the balls machined, in order to remove all pits and rust spots. On being re-assembled the joints were tested to 300 lb. It was then found that after being subjected to test pressure the flexible joints could not be rotated until the ball had been forced back into its original position, for, under test pressure in the shop, the ball, although a perfect sphere, would compress the lead as much as $\frac{3}{16}$ in. and lock itself. This would tend to bear out the theory that, in laying a flexible joint line, a tighter line can be obtained by stretching it by bolting on a head and applying pressure at frequent intervals in the course of laying.

The 24 and 28-in. pipes were 100 ft. apart for the greater distance between harbor lines, but at the east line the two pipes were only 10 ft. apart. For the new location of the 24-in. pipe it was necessary to dredge 18 ft. below the 28-in. line and still maintain the pipe in service. This was accomplished successfully by driving a pile bent over the pipe every 16 ft. and supporting the pipe in cable slings.

In relaying the 24-in. pipe the initial connection was made in 20 ft. of water by a diver, and in such good time that it was proposed to bolt up six lengths of pipe on the barges and have the diver make an under-water connection in each case. The fourth connection under this system required 4 days, and it was then decided to make the connections above the surface. By using graduated slings on a bridle it was possible to lay the pipe without straining the flexible joints which were placed at intervals of 40 ft. The diver released the slings as the pipe reached its final position in the trench.

Mr. Morrow.

To handle this pipe successfully it required three floating derricks, operated side by side, and one small barge derrick for shifting the pipe. This method of laying proved very successful, and a crew, consisting of one foreman, nine laborers, a diver and helper, with four floating derricks, laid this pipe from harbor line to harbor line at a cost of $2.04 per ft.

Tests on this line as relaid showed a leakage of 2 100 gal. per day, 10 months after the completion of the work.

Removal of 28-Inch Line.—The removal of the 28-in., cast-iron pipe, after the completion of the 24-in. pipe, was an easy matter. Due to the type of joint, this pipe can be hauled out of the water like a cable—the lead and the cast-iron yoke lock the pipes together so securely that, after the first lengths had been broken away and raised to the surface, the diver was not needed to put on the slings, as they could be dropped down under the pipe to a point below the ball, and the 45-ton floating derrick supported the load while each pipe was disconnected. It was necessary to melt the lead joint with a blow torch as the flange on the socket of this pipe extends several inches beyond the center of the ball. Melting out the joint was slow work, and the time saved by the method used in raising the pipe was lost in disconnecting on the barge.

The cost to the contractor of taking up this pipe was $2.00 per ft. for a foreman, seven laborers, and two floating derricks.

This pipe had been laid for 20 years, yet was in very good condition when taken up. A slight cut taken off the balls will put it in condition for relaying at some future time.

Such figures as to the cost of removing and relaying these pipes as are given are based on work done between harbor lines under the most favorable conditions, with no sudden or material changes in the stage or currents of the river, which was a few feet above low water; nor do these figures take into consideration the cost of the work outside of harbor lines, which was about three times as expensive as the work on the river. The expense to the contractor for breakage to the cast-iron pipe and yokes, due to the method of taking up the 28-in. pipe, is not considered in the cost figured for this work.

Mr. Herschel.

CLEMENS HERSCHEL,* M. AM. SOC. C. E. (by letter).—When it is considered that any riveted or screw-jointed pipe line, of whatever diameter, has a certain radius to which it may be bent without causing leakage of the contents, it raises the question why this kind of pipe should not wholly replace hinge-jointed pipe lines.

If such a steel pipe line, though even of tunnel dimensions, be loaded on the exterior, or interior, or both, so as to give the combination a specific gravity slightly greater than 1.0, and the front

* New York City.

end be then closed, water-tight, the pipe may easily be hauled across on the natural, or on a prepared, bottom of the river, to which it will fit itself by virtue of its inherent flexibility. Mr. Herschel.

Weighing little or nothing in the water, it exerts little or no pressure on the bottom on which it slides; and, without material pressure, offers no material frictional resistance.

Heavy pipe, or pipe full of water, may also be hauled across on the bottom. Sixteen lines of 12 and 18-in. flexible-joint cast-iron pipes have been thus hauled across a 1 000-ft. channel, at Vancouver, B. C.;* and quite recently the technical press has contained accounts of the hauling of about a mile of pipe out from a sand beach into deep water, off Tampico, Mexico, to enable tank steamers to load with oil.†

In 1895 the writer laid seven lines of 18-in. screw-jointed pipe (the largest size of welded pipe then made) by the method described, in a trench dredged for the purpose, across the Passaic River at Belleville, N. J., at a narrow crossing that was constantly impeded with passing vessels. Flexibly jointed pipe could not have been laid at this place from floats or scows.

A flagstaff set on the forward end indicated where that end was at any moment; and the rear end was added to, and extended backward, on a kind of launching way, as the pipe passed into the water from that end.

During the passage of the forward end across on the bottom of the river, filling the pipe with compressed air furnished proof that there was no leakage; and for 19 years a Venturi water meter, set in this line of pipes, on each shore, has proved that they do not now and never have leaked, though under some 350 ft. head. On the other hand, it is doubtful if any form of flexible-joint pipe ever laid will stand, or would have stood, such a test equally well, it being remembered that water under 350 ft. head will cut out lead very rapidly if a leak is once started.

A 6-ft. pipe was laid under the Hackensack River by this general method some years later. The fact that it broke apart at one point at the bottom of the pipe, shortly after it was laid, is no criticism of the general method of laying such pipe, here described, but only of the special method of loading the pipe in this instance. The break was immediately repaired, and the pipe is to-day doing good service.

The loading must be designed so that it will not affect the natural flexibility of the pipe, which is far greater than one would suppose. Speaking from memory, a riveted 4-ft. pipe can be made to assume a curvature of 1 000 or 1 200 ft. radius, without causing the pipe to leak; and such a radius is easily attainable in a river cross-sectional profile; or the pipe may be built on a curve following the profile across

* The method is described in *Engineering Record*, September 6th, 1913.

† *Engineering News*, Vol. 71, pp. 1078-1080 (1914).

Mr. Herschel. the river selected, a special curved launching way being used when hauling the pipe across.

The pipe weighing almost nothing in the water, 10 000 ft. of it, or any other length, may be hauled across with almost no greater effort than that required to haul 100 ft. In a design made by the writer for crossing the Kill van Kull, separating Bayonne, N. J., from Staten Island, the loading of the pipe was made to take the form of cast-iron shoes or sleds, all connected by long links so that no pull would come on the pipe itself when hauling it across. The outside of the pipe may be protected against corrosion by repeated wrappings of gunny cloth and liquid asphalt.

In the writer's opinion, the general method here described could be used advantageously in the construction of subaqueous tunnels; but this, maybe, "is another story".

Mr. Allen. KENNETH ALLEN,* M. AM. SOC. C. E.—The speaker would like to ask Mr. Wiggin whether, with such a finely machined surface in the joint, by which any bending load is transmitted directly through the metal of the pipe, it would not have been practicable to use, instead of lead, some composition having a coefficient of expansion by heat of zero, and thereby do without caulking?

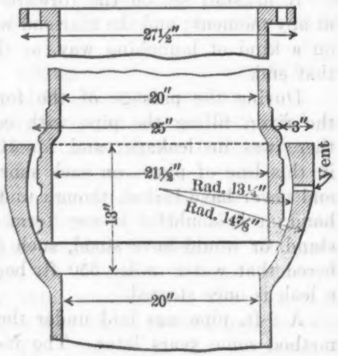


FIG. 5.

At Atlantic City, there was a 20-in. submerged pipe crossing Beach Thoroughfare and a 12-in. pipe crossing Inside Thoroughfare, both tidal streams. The former consisted of three lines of bell-joint pipe, two of which were nearly laid before the speaker took charge of the work. The third line replaced an old 20-in. line, and was similar, except as to the details of the joint, which was leaking and in a generally bad condition. Fig. 5 shows the new joint. One difficulty experienced with this joint was that when, on laying, it was deflected sufficiently, one of the projecting rings on the spigot end came so near the bell that there was not sufficient clear space for effective caulking by the diver, in case of leakage. This difficulty might be obviated by having the projecting rings on the inside of the bell, as in the Portland design.

In taking up the old line, several joints were pulled apart by the tension in lifting. The new lines were laid on cast-iron cradles resting on sills. Each sill was bolted to two piles, forming a bent, which

* New York City.

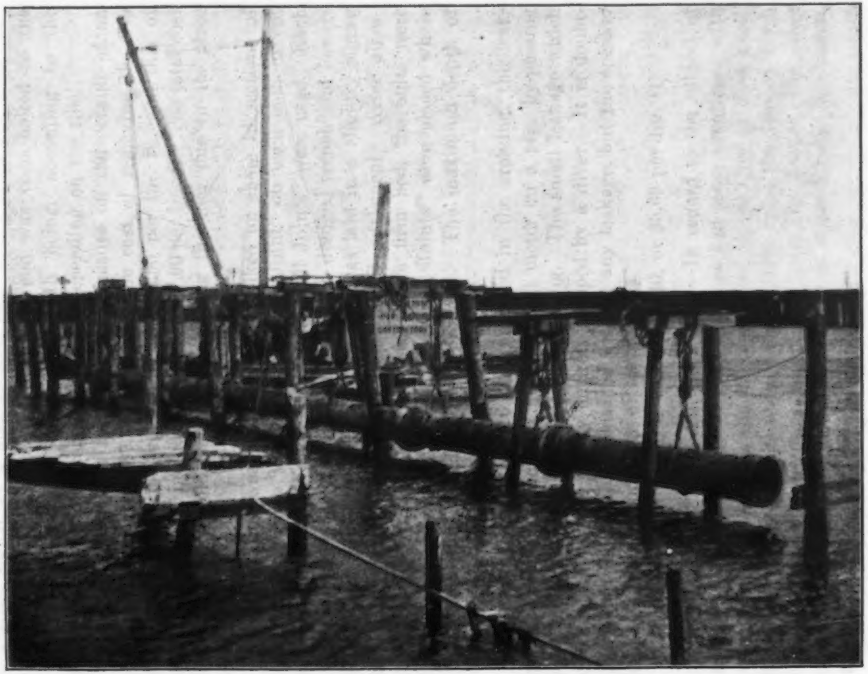


FIG. 6.—LAYING PIPE AT ATLANTIC CITY.

Mr. Allen.

were afterward cut off below water. The length of the crossing was about 240 ft., and the bents, shown on Fig. 6, were 12 ft. apart. The pipe was caulked in sections of six or seven lengths above water. Each section had a flange at each end and was lowered a certain distance, on a given signal, by a man at each bent with a block and fall, until it rested on the cradles. The flanged end was then bolted to the previous section by a diver, the depth being, according to the speaker's recollection, from 12 to 16 ft., depending on the tide.

The pipe, bought as an emergency measure on the default of an earlier contract, cost $87.50 per ton. The cost of removing the 240 ft. of old line of submerged pipe was $2.90 per lin. ft., and that of laying the new pipe, including dredging, $7.90 per lin. ft. The total cost of taking up the old main and relaying the new one in its place was $9 762.

The 12-in. line was made up and caulked on shore in sections of three lengths with a Walker "Half Ball Joint" on each end, except at two places where Walker "Universal Ball Joints" were used. Each section was then lowered from a scow to a dredged trench and loosely bolted to the previous section. The line was laid in a slightly zigzag position to permit a reasonable amount of settlement. After allowing a certain time for the pipe to find a firm bed, the bolts were tightened up by a diver. The "Universal Joints" were placed where further deflection would be most probable. The maximum depth of water in this case was nearly 30 ft.

After completing the 316 ft. comprised in the crossing, the leakage was tested by placing a ¾-in. Trident meter on a pipe by-passing the valve at the supply end of the crossing. The small leakage which occurred at first was finally entirely stopped by a diver. It is doubtful whether the line has remained without any leakage, but the speaker had no further opportunity of testing this.

The total cost of this line was $1 799.40, or $5.69 per lin. ft.

Mr. Kellogg.

R. C. Kellogg,* Jun. Am. Soc. C. E.—In regard to the subject of caulking pipes under water, the speaker has had some experience with two gas mains laid across the Harlem River, in the City of New York. One of these was a 20-in. main, laid in 1890, from the foot of 129th Street and Second Avenue to Lincoln Avenue, The Bronx. The other was a 24-in. line crossing the river between 135th Street, Manhattan, and Mott Avenue, The Bronx.

The method of laying these lines was practically the same as that described in the paper and in some of the discussions, that is, by making the joint above water and lowering the pipe to the bed of the river with the derrick. There was this difference, however, the

* Brooklyn, N. Y.

Mr. Kellogg. joints were made up of yarn and poured lead on the scow derrick and, after being lowered into the water, were re-caulked by divers by hand. This was possible because the pipes were permitted to rest on heavy timber blocks laid immediately back of each hub. The pipe had Ward flexible joints and the hub was of such a diameter that the spigot could be entered rather freely.

In the article* by the late Emil Kuichling, M. Am. Soc. C. E., referred to in the paper, a page is devoted to illustrations of joints used in submerged pipe work. Fig. 13 on that page shows the joints used on these gas lines.

These mains have shown no leakage. The 20-in. line has been broken three times, to the speaker's knowledge, by the dredging operations necessary to keep the channel in the Harlem River to the depth prescribed by the Government.

When these mains were laid, the Gas Company took chances on this very contingency, because they were placed in a shallow trench formed by blowing off the top surface mud with a jet from a hose.

One speaker has referred to the effect of the blow of the hammer in caulking joints in pipes, with special reference to the distance back from the face of the joint in which the impact of the blow is effective in compressing the lead. For 7 years the speaker has used hot lead and lead wool in caulking joints in gas mains, and in that time, has had occasion to examine the effect of the blow of a hammer on hot lead and also on the comparatively new product, lead wool. Several joints of each kind have been examined by sawing through the joint, thus obtaining a section exposing the yarn and lead. Where hot lead is used, in no case has the compacting effect of the blows of the caulking hammer extended back more than $\frac{1}{2}$ in. from the face of the joint. Furthermore, when poured into a joint, hot lead contracts on cooling, and draws away from the inside face of the bell, leaving a space of from $\frac{1}{64}$ to $\frac{1}{32}$ in. in width.

In caulking with lead wool, the process is cumulative, that is, the joint is made by driving in successive strands of lead until the caulking space is entirely filled. Cross-sections of joints made with lead wool show a solid piece of lead from the face of the joint to the yarn, and the first strand driven in has the additional effect of compressing the yarn considerably. The speaker has laid gas mains under the Harlem River in which the joints were made exclusively with lead wool.

The discussion of the paper by members connected with the New York Board of Water Supply and engaged in constructing The Narrows Siphon, is very interesting; the form of joint adopted for that work shows great ingenuity, and is a credit to them.

* *Engineering and Contracting*, April 15th, 1914.

Mr. Kellogg.

It is unfortunate that the depth of water, tidal currents, and commercial tonnage passing through The Narrows, together with its exposed condition, made it impossible to have the work done by divers. The speaker would like to suggest, however, that there is a diving apparatus now manufactured for which the makers claim a possible diving depth of 200 ft., and investigation of its merits might prove of interest in case the present scheme proves unsuccessful under working conditions.

Mr. Wiggin.

THOMAS H. WIGGIN,* M. AM. SOC. C. E.—The author refers to the scarcity of information about submerged pipe lines of the larger sizes. In connection with a 36-in. line which the New York City Board of Water Supply is laying across The Narrows, the speaker had occasion to look up articles on submerged pipe lines, and can vouch for the author's statement, particularly with reference to the completeness of the records. Thanks are due to the author, and to Messrs. F. and A. S. Riffle, who wrote the previous paper, for supplying such valuable information on the large Portland submerged pipes. The speaker would suggest, however, that Mr. Clarke make a few additions which will fill some of the wants felt by the speaker when looking for data on such pipe lines.

Prices, for example, are given sometimes in lump figures. A price per pound, or details permitting easy translation to price per pound, would be of much more general use. In connection with the dredging, the price given is so much per cubic yard of material. A question immediately arises as to how the material was measured. One of the main problems in specifying for The Narrows siphon was how wide the bottom of the trench should be, what the side slopes should be, and whether dredging should be paid for by place measurement or by scow measurement; in other words, whether a great deal of latitude should be allowed in the width of the trench, and the City be made to take the risk as to whether the quantities would over-run, or whether a particular width should be prescribed and merely theoretical quantities be paid for.

There is also the question of the uniformity with which the bottom can be dredged. For example, is it practicable to dredge the bottom within 6 in. of a given plane, as has actually been attempted, or must an allowance of several feet be made, so that the pipe line will go up and down, which, being a flexible line, it is able to do? Those details, as to the actual success in doing the work, would be helpful.

The exact method of making up and caulking the joints is of great importance. The joint used at Portland is what is commonly called a Fanning joint, from the originator of the type. It will be noticed in the illustrations that it is divided by a flange on the great

* New York City.

Mr. Wiggin.

circle. That enables it to be made up in two parts; in other words, the bottom of the joint can be poured and caulked, and then the outer part of the joint can be treated in a similar manner, thus producing two zones of compacting for the lead; that is, two rings of caulked lead. It is important to know how the work was actually done.

The extent of under-water caulking is also a very important matter in a long line of this sort; that is, was it generally made permanently tight at the surface and then sunk to its position in the trench, or was it caulked again after it was submerged. This touches on an important point, namely, how well a flexible joint will stand being deflected, and how tight it will remain after it has been deflected. In some of the experiments in connection with the development of a design for The Narrows siphon, caulked joints were tested before deflection, and then deflected and tested again, and it was found that the movement of the joint caused it to be about as leaky as it was before it was caulked originally.

The particular type of joint used at Portland may be more successful in that respect, if it was caulked in two parts; for example, if the center part of the joints—that part on the great circle—is actually caulked. The joint tried on The Narrows siphon experiments was not caulked on the great circle, but at the outer point of the joint. It was the so-called Duane or modified Ward joint.

The depth to which caulking extends is surprisingly small. In some experiments by the Board of Water Supply, portions of a joint like the Duane were made up with pieces bolted together, so that after caulking they could be unbolted and taken apart; it was found that, in a joint about 6 in. deep, the only part that was affected by very vigorous caulking with pneumatic hammers was about $\frac{3}{4}$ in. at the face; the remainder of the lead was so far from being in contact that a small wire could be run between the lead and the iron. It is evident, therefore, that the method of caulking and the location of the zone on which caulking is done is very important, and particularly the question as to how much caulking will be required under water to produce a finally tight line.

Reference has been made to the effect of pull on joints, and it will be perfectly evident, the speaker thinks, that a pulled joint is bound to be tight while it is being pulled; the condition is that of a sphere being pulled out of an enveloping sphere which is too small at the opening to permit the escape; and the escape is being prevented by a band or zone of lead which is squeezed between the inner and outer spheres. The harder the squeeze the more water-tight the joint will be. The question is, will it stay tight after the pulling and resulting squeezing have stopped?

The ideal line might be one that is laid on such a foundation, or say, such lack of foundation, that it will at all times be subject to

a catenary pull by virtue of a slight tendency to settle. Such a line would remain tight. The practical difficulty of obtaining such conditions is obvious, though it is probable they exist in some short lines; but if one imagines a line laid on a level bottom, and made tight by pulling, and then supposes that the temperature of the water surrounding that pipe, and in the pipe, as might be the case, rises 30 or 40°, and each length of pipe lengthens, then the contact would be relieved. The same condition would result from settlements, even slight, at accidental local summits in the line or at changes of grade where the line was convex upward. The only thing that would keep the joints in contact would be the elasticity, not in the lead (which has practically none), but in the iron inside and outside of the joint; that is, in the bell and spigot. If these are stressed sufficiently during the pulling of the joint, and if it is supposed (which in the light of experiments by the speaker is improbable) that the lead will not yield gradually and relieve that stretching and compression in the iron, then the change in length of the pipe due to temperature changes might be compensated for by a contraction of the bell, thus keeping tight the joint which had been made tight by the stretching process. Mr. Wiggin.

None of these questions, however, has been answered from existing literature, because the behavior of submerged lines after having been placed in service is not known; in other words, there are no meters of adequate kind—in fact, generally speaking, there are no meters of any sort—to tell whether the lines remain tight. Only the grosser leaks can be discovered, such, for example (like that described by the author), as would lower a reservoir.

In the pipe which is being laid in The Narrows, the attempt has been made to produce initial tightness, and to overcome the difficulty due to changes in length of pipe with temperature, by substituting for caulking, the filling, by injecting lead with screws, of all shrinkage space in the joint, so far as practicable, and especially on the great circle; then, no matter if the spigot is moved slightly in and out, the joint will still be tight on the great circle, and act as a sort of expansion joint, rather than be tight at the front only and afterward become loose by temperature movements or movements due to settlements in the line.

In connection with the break in the 28-in. pipe and the uncertainty as to the location of the break, the thought occurs that perhaps it would pay to put meters at each end of such a line, in order to discover when it starts to leak more than a slight percentage. A check-valve between the crossing and reservoir would have prevented the emptying of the reservoir. Of course, it may not always pay to make such provisions. On the line across The Narrows, meters and check-valve both have been provided, because it is very

Mr. Wiggin. important that it should be known if the line starts to leak, and also that the water stored in the reservoir should be conserved for use while the line is being repaired.

The break in the Portland 28-in. pipe took place by fracture across one of the pipes, which are nearly 16 ft. long. The paper gives no diagram, but, from the description, the break took a long slant, such as would occur in a cross-grained stick of wood, the fracture being apparently about 12 ft. long. This would indicate that the pipe was perhaps resting on a hard spot in the middle and acted as a beam. Some such explanation is evidently in the mind of the author. It raises the question as to whether 16 ft. is not too long for a cast-iron pipe, particularly on a hard bottom. In the case of The Narrows siphon, it was decided, after considering the Portland 28-in. pipe, which is very well described in the paper by Messrs. F. and A. S. Riffle—a paper which furnished a great deal of useful information in connection with studies for The Narrows pipe—to adhere to the shorter length of 12 ft., partly because foundries are rigged for 12-ft. lengths, but mostly for greater safety in connection with foundations and flexibility.

The new Portland steel lines with cast-iron flexible joints have considerable stretches between flexible joints, apparently more than 35 ft., and the question suggests itself whether the pipe will continue everywhere to span successfully between flexible joints, particularly if the bottom is hard, and in view of the fact that the back-fill under a pipe is never perfect.

Experience with The Narrows pipe proves that it is very difficult to get a level bottom, though the method of dragging the trench bottom is now producing good results. At best, however, there will be occasional sharp changes in grade, due to the working of the pipe-laying cradle in rough weather. Back-filling at such places is likely to cause undue strains in joints and in pipes. Further details of the depth and method of placing back-fill at Portland would be interesting. A depth of from 3 to 4 ft. is mentioned. A photograph in the Riffle paper shows a comparatively deep trench at the shore.

The details of the repair sleeve are very interesting; the speaker, however, would like to have a statement as to how the sleeve was filled. Apparently, it was grouted after being placed. The 8 by 8-in. angles and other parts of the longitudinal joint, if they were required to take the full hydrostatic pressure over the full diameter of sleeve, would apparently be very heavily strained. Was the grout assumed to be so impervious that the pressure did not reach the whole area of the sleeve?

Another point, on which information would be useful, is the accuracy and smoothness of the machine work. In the original studies for The Narrows pipe, much help was obtained from the records of

the 28-in. Portland line in the matter of machining, and an extension of the record to include the new lines would be valuable. In the matter of smoothness, no accurate data were found, because it is difficult for civil engineers to describe machining. It is not impossible, however, to describe the lathe tool, speed, feed, velocity of revolution, etc. Mr. Wiggin.

The success of a flexible joint, to a large extent, depends on how smooth the turned parts can be made. In the work now being done for The Narrows line, not due to the specifications of the city, but to the contractor's energy in the matter and to his appreciation of its importance, the surface is actually being ground, making it equal to good cylinder work, and the results are apparently justifying the method, although, of course, as the line has been only partly laid, it is too soon to say what the final outcome will be.

In experimenting for The Narrows pipe, lead wool, with the different degrees of caulking, was tried. It was easy to get a tight joint, but the trials were not successful in obtaining at the same time the proper degree of flexibility. The experiments were not wholly exhaustive, as the method was much slower than desired and the method of injecting lead mentioned previously proved to be more desirable for flexible joints.

Non-shrinking alloys of lead were also tried, by a company which has all the metals and is expert in mixing them, but it was impossible to mix one that seemed to meet the requirements. This idea looks very promising, and the speaker is now sure that he understands why it was not successful.

W. E. Spear,* M. Am. Soc. C. E.—Mr. Wiggin has brought out many of the points which this paper suggests to those interested in submerged pipes. It would be interesting to know how accurately the trenches at Portland were dredged, and also the final grade and alignment of the pipes. As Mr. Wiggin states, the 36-in. pipe line under construction in New York Harbor was laid at first on a somewhat uneven bottom, but the vertical deflections of the pipes were not great. In addition, the line also deflected laterally at some points, due to the thrust of the pipes in sliding down the launching ways. Similar deflections in the Portland pipe would have exposed that portion of the lead which had been caulked at the face of the bell, so that unless the joints were poured and caulked in two operations, as Mr. Wiggin suggests, it is difficult to understand how the line could have been made tight without submarine caulking after the pipes were finally placed. Mr. Spear.

Some experiments, made in New York Harbor several years ago, on the depth of penetration of large anchors, indicate that the cover

* New York City.

Mr. Spear. of 4 ft. placed over the Portland pipes would not be quite sufficient. A cover of 8 ft. is being placed over The Narrows siphon.

In harbor waters the life of a submerged pipe line is a matter of some concern, and it would be of great value to those in charge of such works to know the condition of the 28-in., cast-iron pipe at Portland when it was taken out after 20 years of service, especially the condition of the finished surfaces of the bell and spigot, and how much work would have to be done on them to make them sufficiently smooth to place again in service. Apparently, they were good enough to permit them to be used again, because they are being stored for that purpose.

The work at Portland was evidently done at a very reasonable cost. The pipe line in New York Harbor is of greater size and is laid at a much greater depth, in a swiftly moving tideway, crowded at times with traffic. The cost of this pipe, on the basis of the quantities originally estimated for several items, would be about $95 per ft., but some economies will be effected in dredging and back-filling that will probably reduce the cost to about $85 per ft.

Mr. Hogan. J. P. HOGAN,* ASSOC. M. AM. SOC. C. E.—It is interesting to note that the pipes at Portland had to be taken up and relaid on account of the necessity of deepening the channel. In The Narrows siphon, which is now being laid by the Board of Water Supply of New York City, the Government requires the City to lay pipes so deep that they will never have to be taken up. The line is about 10 000 ft. long; the minimum depth will be 60 ft., and the maximum, 74 ft.

Mr. Herschel suggests that it might be possible to make up a line with riveted or screw joints and drag it across, but there would be some difficulty in doing that in this case, so that, even if a screw joint or riveted pipe is useful for short distances, a flexible pipe line has certainly a field of its own.

It would be of interest to know how long it takes to make up the Portland joint. In comparison with the Ward joint, which is being used in The Narrows, it seems to be rather clumsy, and might take a much longer time to make it up. In New York, the pipes have been made up and sunk in 1¾ hours per length, for the entire operation, and it does not seem possible to make up the Portland joint, which requires the bolting of a flange, in nearly as short a time, particularly if it is one with the double caulking. In any case it would be valuable to have every detail in regard to the exact method of making up this joint and the time required to do it.

It may be of interest to know that the joint used on the Board of Water Supply work is substantially as Mr. Wiggin has described it; the shrinkage of 10%, which occurs when the lead cools, is made up

* Tompkinsville, N. Y.

by forcing in cold lead through 32 holes by jib-screws. The joint takes about 300 lb. of lead at a pouring, and about 22 lb. are forced in afterward. With mechanical appliances, the operation of forcing in the cold lead takes about 30 min. Mr. Hogan.

On the Portland joint the stop for holding the lead is in the bell, which would indicate that the lead remains fast to the bell and rotates on the spigot, whereas, in the New York joint, the lead remains fast to the spigot where it shrinks, and the bell rotates around it.

On page 1307 it is stated that "the final test, under a pressure of from 170 to 180 lb., showed a leakage of approximately 7 gal. per min., or 10 080 gal. per day, or about 8 gal. per lin. ft. of joint, per day, under the pressures noted, which are approximately 50% in excess of the normal working pressure". In New York, the experience has been that the higher the pressure the tighter the line, so that it would seem that, if the pressure was 50% in excess of the normal working pressure, the results shown are more favorable than would be found under the actual working pressure. In tests of a certain part of The Narrows line at 80, 90, and 100 lb., the leaks were greater at 80 than at 90 lb., and greater at 90 than at 100 lb. That may be due partly to the tightening of the joints at the ends of the line because of the longitudinal pull. With the New York joint, it is due, in a great measure, to greater resistance to pressure of the bell than of the spigot.

Both Mr. Spear and Mr. Wiggin have mentioned the up-and-down character of the New York line. That was a feature at the beginning of the work, because in dredging at a depth of 60 ft., it was difficult to do accurate work on account of the heavy tidal current in The Narrows, which is about 3½ miles per hour. It is also extremely difficult to ascertain the result of the dredging, because it is very troublesome to make soundings in heavy currents at a depth of 60 ft., and almost impossible to make area soundings. Fairly accurate profiles may be made, but it is very difficult to ascertain whether or not the bottom is even. In the first part of the line, especially in attempting to dredge down grade, the bottom was uneven, but that defect was obviated later by the use of a V-shaped drag consisting of two heavy steel members, about 25 ft. long, with a spread of about 20 ft. at the base, across which there is another heavy member with a plate inclined forward to form a cutting edge. The weight of the drag is between 8 and 10 tons, and, by hauling it up and down the trench four or five times, it has been possible to get a very level bottom, so that at present no trouble is experienced from up-and-down variations of the pipe, except in storms. It is impossible to lay pipe during storms in such an exposed position. It is not safe to move ahead, and, at that time, the skidway or cradle in which the pipe is supported tends to work

Mr. Hogan. into the bottom, sometimes as much as 2 or 3 ft. That leaves a hollow or depression which sometimes causes a maximum deflection in one or more joints, but such variations in grade have been straightened out successfully. However, in laying at this depth and under these conditions, it is a question whether a satisfactory result could be obtained with any pipes but those with flexible joints, as one cannot expect to have the bottom of the trench as even as if it were dug on land or in shallow water.

In reference to Mr. Kellogg's remarks, it may be stated that the Ward joint, as used by the Board of Water Supply, has a bell which is large enough to receive the spigot, and has no flanges. Also, that no work is done under water. The joint is made up and made tight on top, and then lowered into place and never touched after it is laid.

Mr. Wiggin has stated that joints caulked with lead wool were tried. Undoubtedly, a lead wool joint can be made up successfully under water, but the effort was to get a joint which could be made up on top, and would be flexible enough afterward to be lowered into place and require no attention after it was laid; it was found impossible to do that with lead wool.

Mr. Brush. W. W. Brush,* M. Am. Soc. C. E.—Messrs. Wiggin and Spear have brought out the more interesting points in connection with the submerged pipe lines at Portland, Ore. An interesting question that has not been raised is how much deviation in line and grade is permitted in submerged pipe line construction? A study of the specifications does not give a satisfactory answer to this question, as they may or may not state definite limitations. Even when definite limitations are given, investigation has shown that the work, as carried out, did not come within such limitations. If Mr. Clarke could give this information for these submerged pipe lines, it would be useful in considering specifications for similar work.

The experience of New York City in repairing submerged pipe lines is similar to that of Portland in regard to the time required. The officials responsible for appropriating funds do not seem to realize the necessity of a secondary feeder for safe-guarding a supply which is dependent on submerged mains. When the engineer states that the repairs to a submerged line may require weeks or months, rather than days, the layman frequently considers this statement to be biased. As a result, proposed pipe lines for secondary feeders are not put in, due to lack of funds.

Within the past year, New York City has had such a case, where one of two lines has been out of service for approximately 6 months, and will be for several months to come, and yet, less than 2 years ago, the authorities refused to authorize a contract for laying a second-

* New York City.

ary feeder main recommended by the engineers of the Water Department, although one line was insufficient for fire protection. Mr. Brush.

L. J. LE CONTE,* M. AM. SOC. C. E. (by letter).—The writer is greatly pleased with this valuable paper. It is full of practical data based on actual experience, which, of course, makes it of particular value to the engineer in charge of water-works. There cannot be fair-minded discussion, however, unless all the local facts and conditions are known. Mr. Le Conte.

The work described is highly commendable in every respect, and the writer has only a few suggestions to make regarding ball-joints. He has had much sad experience with cast-iron ball-joints due to unexpected deflections in excess of the angular limit, 15° in this case. When the deflection is greater than this, the cast-iron ball-joint is likely to "snap", like a glass bottle. The writer, therefore, is firmly of the opinion that the less cast iron there is in a ball-joint the

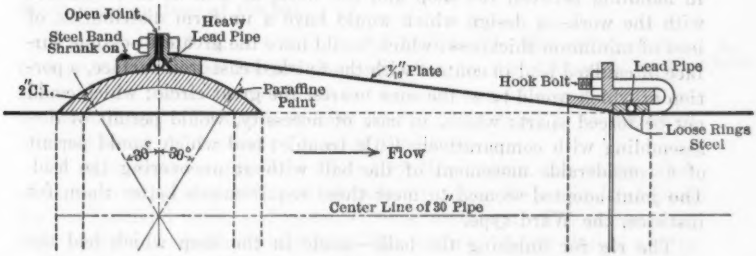


FIG. 7.

safer it is against extreme deflections. Accordingly, he presents a suggestion (Fig. 7) for a ball-joint which has much less cast iron and a greater deflection angle, 30° each way, and, in case there is an extreme jam or pinch, this ball-joint will stand the strain much better. A circular segment of a cast-iron shell is riveted to the extreme end of a pipe section. The socket into which this ball enters is in the form of a steel frustum of a cone, the length of which is about 1.5 times the diameter of the ball. The packing of the flange joints at the two ends may be made to conform to any of the standard methods based on experience. For the 30-in., lap-welded, steel pipe, this ball-joint can be manufactured for $366, but the length of the joint is 7 ft., making the average cost $52.30 per ft. The cast-iron ball-joint described by Mr. Clarke costs $345, but the length is only 3 ft. 4 in., thus making the average cost $103.20 per lin. ft. Mounted on the plain siphon pipe, the average cost of these ball-joints per

* Berkeley, Cal.

Mr. Le Conte. linear foot would be $19.50 for the author's siphon and $18.00 for that of the writer. He believes, therefore, that it would have been quite possible to lay a better and safer siphon for a little less money.

Mr. Ward. C. D. WARD,* M. AM. SOC. C. E.—Mr. Wiggin has mentioned the difficulty of moving a joint made with lead wool. As the speaker is not familiar with the use of this material, he would like to ask how hard the joint was caulked, and, if it had not been caulked so hard, would it have been possible to move it?

Mr. Phillips. W. R. PHILLIPS,† M. AM. SOC. C. E. (by letter).—The selection of the design of the ball joint for the 30-in. submerged pipe line across the Willamette River, described by Mr. Clarke, was governed largely by the great importance of having the work of assembling completed in the shop, for there only could the workmanship be thoroughly inspected.

This involved a design which would be least likely to be injured in handling between the shop and the work, as well as in connection with the work—a design which would have a uniform distribution of lead of minimum thickness; which would have the greatest possible surface of caulked lead in contact with the finished cast-iron surface, a portion of which would be at the zone nearest the great circle; which could not be forced apart; which, in case of necessity, would permit of dis-assembling with comparatively little trouble; and which would permit of a considerable movement of the ball without uncovering the lead. The joint adopted seemed to meet these requirements better than, for instance, the Ward type.

The rig for finishing the balls—made in the shop which had the contract for furnishing the joints—was attached to the ways of an ordinary lathe. It consisted of a swinging arm which carried a tool rest pivoted under the point where the axis of the ball intersected the plane of the great circle. A tool set on a level with the lathe centers would swing horizontally in an arc of the same radius as that of the great circle, and thus insure a spherical turning of the ball. The tool was fed to its work by using a link connecting the swinging arm with the lathe carriage, so that the feed could be handled in the same manner as in straight lathe work. The speeds used, and the cuts, were the same as in any work of similar size and material. Great care was taken that the finishing cut should leave the work true and smooth, and the lathe tool alone was used to finish the surface of the ball. Tests showed that the balls were practically perfect spheres.

In making up the joint, the socket was set level, with its flange resting on a blind flange. The ball was set into the socket so as to rest on the ridge shown on Fig. 1. The lead was poured, and a

* New York City.

† Portland, Ore.

light sledge was used in caulking. A ⅜-in. tube of heavy rubber was used as packing between the flanges. The ring was leaded and caulked in the same manner as the other piece. When the work was completed, a blind flange was placed on the upper flange of the ball, and the joint was subjected to a test pressure of 225 lb. per sq. in., which pressure it had to withstand without sign of leakage. It was noted that as the pressure was released, the ball settled perceptibly into the socket, thus assuring the freedom necessary for rotation without damage to the lead. Mr. Phillips.

After the joint was tested, nothing more was done to it, as it was then considered to be finished. It is probable that the pipe extended under pressure, and took up the play which developed when the joint was tested.

In lowering the pipe into its bed, care was taken that the sections—each 60 ft. long between the joints—were not deflected with relation to one another to such an extent that the lead would pass off from the finished surface of the ball.

D. D. Clarke,* M. Am. Soc. C. E. (by letter).—The writer desires to express his thanks for the interest taken in the discussion of his paper by the several members of the Society who have commented on the work done in Portland, or have offered suggestions as to improved methods based on experience in other localities. Mr. Clarke.

In reply to the suggestions offered by Mr. Wiggin, the writer takes pleasure in supplying additional details regarding the methods adopted for the Portland work, and measurements, costs, etc. The omission of these details from the paper was due mainly to the desire not to encumber it with matters which were thought to have only a local application, and hence would not be useful in other places. For the benefit, therefore, of those who wish additional information on the several points which have been raised, the following data are submitted.

Mr. Wiggin states:

"Prices, for example, are given sometimes in lump figures. A price per pound, or details permitting of easy translation to price per pound, would be of much more general use."

As a rule, the prices given for the ironwork were those quoted for the several units named in the contract. In the case of the 28-in., cast-iron pipes laid in 1894,† the payment was on the basis of $31.4275 per lin. ft., the weight or price per pound not being given.

In an article‡ by the Ohio Pipe Company, of Columbus, Ohio, the contractors for the manufacture of the 28-in. pipes, describing the process of manufacture, the statement is made that the estimated weight of each pipe section, with sleeves bolted on, was 10 000 lb. The

* Portland, Ore.

† *Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 257 (1895).

‡ *Iron Age*, January 10th, 1895.

Mr. Clarke. actual weight of these joints, as found when the line was removed, approximated 9 628 lb.

Assuming the latter weight and a laying length of 15 ft. 8 in., gives a price of $102.28 per ton of 2 000 lb. for pipes and sleeves; and the contract price for pipe-laying, $2.50 per lin. ft., is equivalent to $8.14 per ton of 2 000 lb.

For the 24-in. line, the sixty flexible joints were contracted for at $260 each. The calculated weight of these joints, including bolts and lead, is 5 365 lb.; the price per pound, therefore, approximates 4.84 cents.

The pipe used for this line was ⅜-in., lap-welded steel with cast-iron flanges at each end of the sections, approximating 20 ft. in length.

Estimated weight of pipe per linear foot	94.62 lb.
Estimated weight of two flanges per foot of pipe	53.00 "
Total weight	147.62 lb.
Contract price per foot of pipe and flanges	$8.50
Estimated cost per ton, therefore	$115.18

For the 30-in. line (submerged portion), thirty flexible joints were purchased for $345 each. The weight of each joint, ready for laying, is 5 720 lb., the estimated price per pound being 0.0603 cents.

The 30-in., lap-welded, steel pipes, galvanized and asphalted, with cast-iron flanges, cost $14 per lin. ft. These pipes were made of $\frac{7}{16}$, $\frac{1}{2}$, and $\frac{9}{16}$-in. plates, the average length of each pipe being about 16 ft. Of this pipe 2 081 lin. ft. were purchased.

Total weight of pipe, including flanges	464 469 lb.
Weight of 3 500 bolts with hexagonal nuts	17 730 "
Total weight	482 199 lb.
Average weight per foot	231.7 "
Average cost per pound	$0.0604

The 30-in., cast-iron pipes used for shore connections cost $11.10 in place in the trench. The estimated weight, per 12-ft. section, is 4 700 lb.* The estimated price per ton of 2 000 lb. is $56.63 for pipe in place, including lead and oakum.

Mr. Wiggin asks how the excavated material was measured.

When the 28 and 24-in. lines were laid originally, the trench was excavated with a ladder-dredge, the excavated material being deposited on barges alongside the dredge. The barges were towed away later, and

*Class "F", New England Water Works Assoc. specifications.

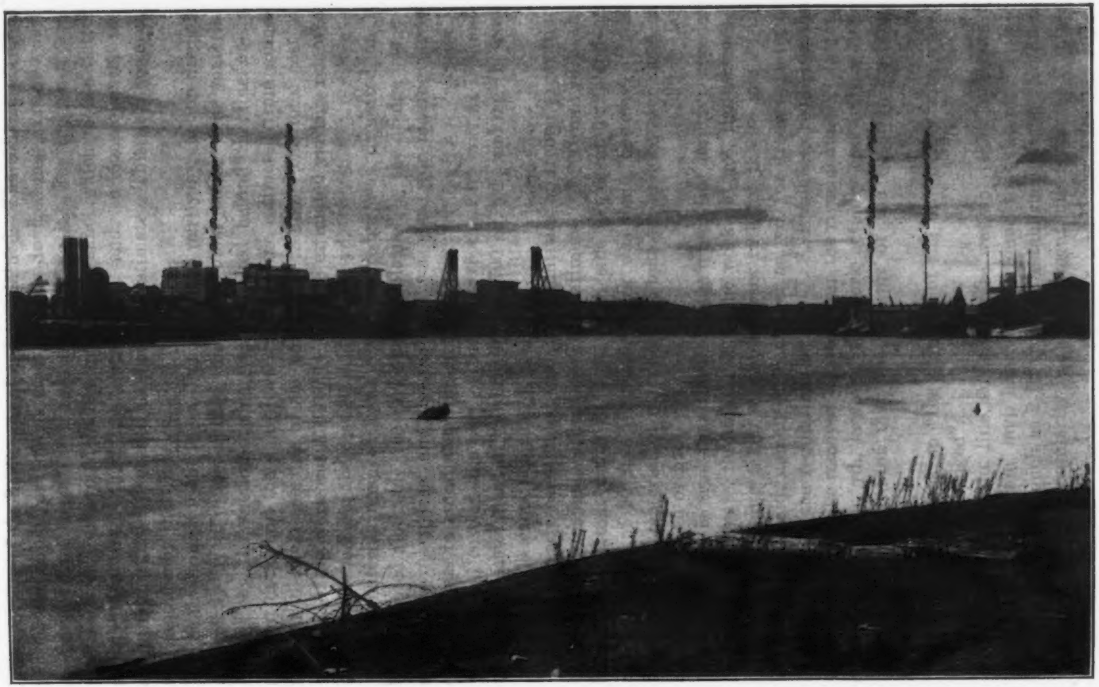


FIG. 8.—UPPER PORTLAND HARBOR, LOOKING NORTH FROM LOWER END OF ROSS ISLAND. HAWTHORNE BRIDGE IN CENTER.

Fig. 2.—Upper Portland Harbor, Looking North from Lower End of Ross Island. Hawthorne Bridge in Distance

the dredged material was used for filling behind the dock line in the lower portion of the harbor, as already explained. The contractors were paid by the cubic yard of material actually dredged and carried away, as measured on board the barges transporting it. For the most part, the trench was excavated in fine gravel and sand, which took a flatter slope under water than was anticipated, and, in consequence, the dredging quantities exceeded the original estimate. This risk, however, was assumed by the City. Mr. Clarke.

The dredging for the 30-in. line was done partly by suction-dredges and partly by a dipper-dredge, on a *per diem* basis. The material removed by the suction-dredge was deposited behind the line of an adjacent dock, and the quantities dredged were estimated from soundings taken along the line of the completed trench. The total cost of this part of the work was $16 160.14, and the estimated yardage removed was 102 655, making the cost $0.1574 per cu. yd.

The material removed by the dipper-dredge was deposited on a barge moored alongside, the barge being unloaded by the dredge owners at their own expense, and the material was used for making street embankments in an adjacent district.

The total cost of this part of the work to the City was $4 158, and the scow measurement of yardage was 29 700 cu. yd., making the cost to the City 14 cents per cu. yd.

It has already been stated that the quantities of trench excavation exceeded the original estimate. This estimate (57 000 cu. yd.) was based on a bottom width of 6 ft., with side slopes of 1: 2, and an average depth of 20 ft. When the work was done, however, it was found that the material in places would take an under-water slope of approximately 1: 5 or 1: 6.

Although it is true that the dredging quantities largely exceeded the original estimate, the total cost of the work did not, the estimated price per yard being taken at the contract rate for the 28 and 24-in. lines previously laid, namely 40 cents per cu. yd. for dredged material and its disposal (the regulations of the U. S. Engineers requiring the trench material to be removed entirely from the river channel). Instead of 40 cents per cu. yd., however, the work actually cost only a little more than one-third of that, on account of the superior dredging facilities, etc.

Mr. Wiggin brings up "the question of the uniformity with which the bottom can be dredged".

In laying the 28-in. line, sufficient care was not taken to secure a uniform trench bottom, and, later, it was found that the pipe as laid varied in places as much as 2 ft. from the established grade. No harm resulted from this irregularity in the alignment, other than as it might

Mr. Clarke. affect the flow of water, for the flexible joints allowed the pipe to adapt itself to the ground surface.

With the 24 and 30-in. lines, greater care was taken to secure a uniform trench bottom, but it is doubtful whether it came within the 6-in. limit named by Mr. Wiggin. This would be extremely difficult to accomplish in a tidal stream, with a depth of water ranging from 20 to 40 ft., or more.

Mr. Wiggin states that "The exact method of making up and caulking the joints is of great importance".

The joints used permitted the lead to be caulked in two rings, and this method was followed.

In the case of the 24-in. steel pipe, laid in 1898, the specifications provided for the following procedure in setting up and testing flexible joints:

"The balls and sockets will be put together in the shop and subjected to tests by first filling with water and afterward by hydrostatic pressure up to 250 lb. per sq. in. In putting the flexible joints together, the socket will be first put on its end, small flange down, and then the ball will be inserted and fairly raised on the projection of the socket, previously bored to the proper size and angle. Then a fibrous gasket will be inserted and driven to the bottom of the cavity, when the melted lead will be poured from as many places as may be necessary to insure a solid body without seams or blow-holes.

"After the setting of the lead, the ball will be raised to ascertain the perfection of the work, and if found deficient it is to be made over. If found perfect, the ball will be replaced and the caulking done in the usual manner; then another gasket inserted as shown, the main gasket put in its place and bolted, but not quite home, when again the melted lead is poured into the cavity. To insure the perfection of the work, this main flange will be firmly bolted home and the final caulking done. If it is deemed necessary to warm up the parts in order to secure perfect work, it shall be done, at the request of the inspector. If during the process of testing any defects shall be manifest, all such shall be made good, and if required new parts will be substituted in place of the defective ones, all at the contractor's expense. While under test the ball will be moved from side to side and thus ascertain if it fits perfectly in all positions when in place.

"To prevent end strains on the ball-and-socket joint while under hydrostatic pressure, the caps shall be held in place by rods reaching from one to the other. The joints shall remain perfectly tight while the pressure of 250 lb. per sq. in. is being applied, and shall so continue until the pressure is entirely removed."

For the 30-in. steel pipe, laid in 1910, a similar process was followed, the specifications providing as follows:

"The ball shall be machined truly spherical and shall have a smooth surface when finished. All flanges shall be faced flat, the end flanges finished for gaskets. When assembled the two parts of sockets shall match evenly and the space for lead shall be practically true to ball.

For making the lead joint, lead wool shall be used, and each part of socket shall have lead joints made as a separate operation with strands at joints between castings. When finished each complete connection shall be subjected to a hydrostatic pressure of 300 lb. per sq. in. after having the angle of surfaces of end flanges moved at least 10° from relative position occupied by them when lead joint was made, under which pressure there shall be no leaks." Mr. Clarke.

From the foregoing, it will be seen that provision was made for two rings of caulked lead for each joint, and the work was done in this manner. The flexible joints, however, were not moved or deflected while being made up.

In the case of the 30-in. line, it was proposed at first to use lead wool, but this was subsequently abandoned.

A question is also asked as to the extent of under-water caulking required.

The provisions for making up and testing the joints were carried out, and, as a rule, no large amount of caulking of lead joints was required after the pipe had been laid in the trench, although the pipe was carefully gone over by a diver, employed by the City, before it had begun to be covered by the sand and silt carried by the current along the bed of the river.

With the 28-in. line, laid in 1894, a pull on the joints was made in the process of laying, as was described in the paper by Messrs. F. and A. S. Riffle.

With the 24 and 30-in. pipes, resort was not had to such a method of producing tight joints. Probably the balls were turned or machined more uniformly, but, at all events, the larger leaks found after the pipe was laid were easily caulked. It should be stated, however, that there were doubtless small leaks which were not located, due, in part, to their being covered before the final test was made.

Reference has already been made to the volume of leakage from these pipes. When the 28 and 24-in. lines were laid originally, a small force pump was used for the final test for leakage. This pump took suction from a vessel of known capacity and, in this way, the volume of water required to maintain the pressure at the desired standard was measured.

In later tests, made when two lines were in use, a small by-pass was inserted between the two mains, and a disk meter was installed to register the flow from the main in service to the one being tested, and in this manner the quantity required to maintain normal pressure in the main being tested was determined.

In the case of the 24-in. line, when first laid, the leakage approximated 10 080 gal. per day, the distance between the East Side and West Side gate-chambers, where stop-valves were set, being 2 706 ft.,

Mr. Clarke.

comprising 2 042 ft. of 24-in., flexible-joint, steel pipe and 664 ft. of 32-in., standard, cast-iron pipe laid on shore above water level.

For the 30-in. line, laid in 1910, comprising 2 265.6 lin. ft. of standard 30-in., cast-iron main and 2 047 lin. ft. of 30-in., lap-welded, steel pipe, with twenty-seven flexible joints (or a total distance of 4 312.6 lin. ft. between gate-valves), leakage tests were conducted in the manner already described, the quantity of water required to be pumped into the main, in order to maintain the desired pressure, being measured by meter. In this case, the first test did not result as favorably as was expected, the total leakage, under the normal pressure of approximately 150 lb., being about 30 000 gal. per day. Later tests, however, showed the leakage to be 5 000 gal. per day. After the 24-in. line was taken up and relaid, the test indicated a leakage of 2 100 gal. per day, which is regarded as a favorable showing.

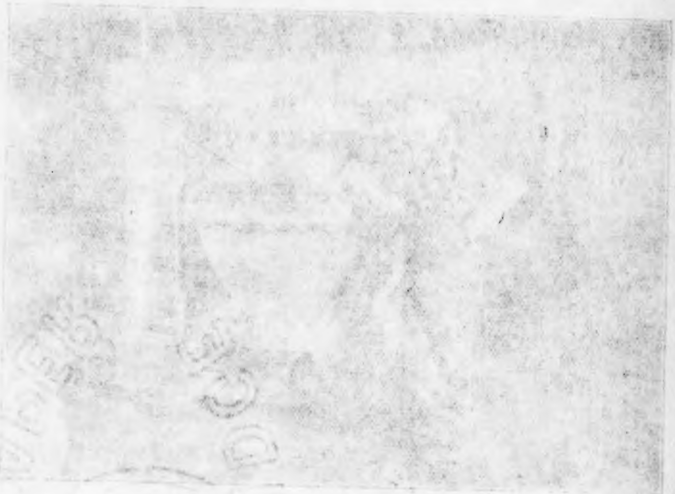
Regarding the attempts which were made to reduce the volume of leakage by recaulking the joints under water, it may be stated that, after the pipe had been laid in the trench, and while it was under pressure, a diver examined all the joints thoroughly and recaulked them when necessary. This examination was made, for the most part, before the silting up of the trench had begun, but, at some points, it was not possible to do this before the pipe had been partly covered by the fine silt moving along the bed of the stream or breaking down from adjacent banks.

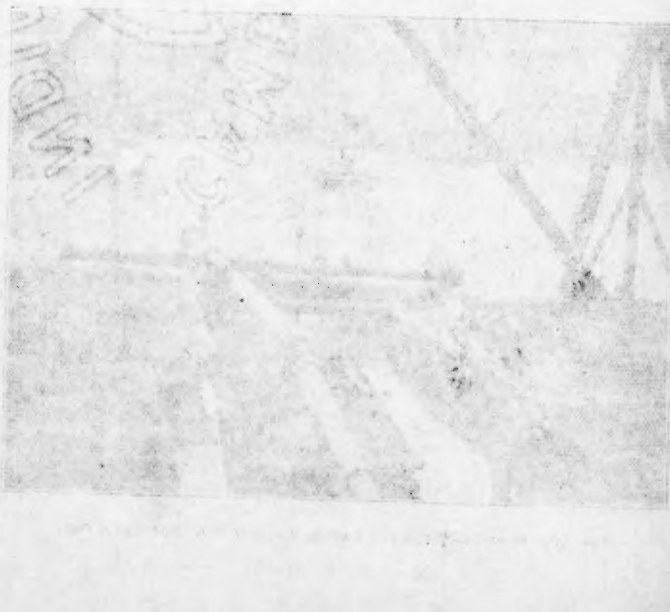
Mr. Wiggin raises an interesting question regarding the depth at which lead joints are affected by caulking, and also as to the effect which the pulling of the joints may have on their permanent tightness. The 28-in. line, laid under conditions which seemed to demand a pulling of the joints, has now been removed, and it is not possible to secure additional data regarding its water-tightness and the effect of the treatment to which it was subjected. The 30 and 24-in. lines were not handled in this manner, but the writer does not know of any way to determine how the joints are affected by temperature changes. On the assumption that the leakage would be greatest during warm weather, when there might be a slight expansion of the pipe due to the higher temperature, a test of the leakage can be made and compared with a similar test made in the winter, but the writer is doubtful as to the results which might be obtained. The temperature of Bull Run water ranges from a minimum of, say, 32° in the coldest weather to an average of from 58° to 62° at the City's reservoirs in the summer, as it flows from the conduit.

The suggestion that meters should have been placed at each end of the submerged pipe lines is worthy of note. In fact, that is what has been done with the new conduit, No. 2, built in 1910-11, leading from the head-works to Mt. Tabor. A Venturi meter was

FIG. 9.—TEST OF 30-INCH FLEXIBLE JOINT, PORTLAND, ORE.

FIG. 10.—DERRICKS USED FOR LAYING 30-INCH PIPE, PORTLAND, ORE.





placed at each end of this conduit for the purpose of determining whether there is any variation in the flow. A leaky joint would hardly be discovered in this way, but a large break would be detected at once. Mr. Clarke.

The fracture in the 28-in. pipe is well described by Mr. Wiggin as having taken a long slant, "such as would occur in a coarse-grained stick of wood", and the most probable explanation of the break is that the metal of the pipe was resting on a hard spot in the pipe trench.

It may be said that, in laying the 24-in. line, rock was encountered at this point, and, in order to avoid blasting in close proximity to the 28-in. line, it was necessary to lay the second pipe close to the first—in the same trench, in fact. Therefore, it is considered more than probable that some projecting rock in the bottom of the trench, together with a heavy overlying embankment, caused the fracture of the 28-in., cast-iron pipe, which has been described. Fig. 11 gives an idea of the appearance of the break.

It has already been explained that when the pipe was uncovered, the two fractured pieces were found to be separated. The drawing of the two ends sufficiently close together to permit of their being enclosed in a steel sleeve did not prove to be an easy matter, and was not accomplished in an entirely satisfactory manner. The results proved that the wrapping of tarred canvas was not effective in preventing the cement grouting from finding its way through the joints in the layers of canvas and into the interior of the pipe.

When the 28-in. pipe was taken up, it was found that the cement had been deposited on the bottom of the pipe to the depth of several inches, but, fortunately, it was not in sufficient quantity to restrict the flow of water appreciably.

Concerning the purpose of the grouting and the benefit derived therefrom, it may be said that it was probably not worth its cost. The main thought was that the cement might aid in the caulking of the joints of the sleeve, and would also encase the pipe and hold it in position.

As has been stated, lead wool was used in caulking the circular joints of the sleeve around the pipe, but the longitudinal seams were packed with a rubber gasket. The probability that the grouting would be so impervious as to reduce the pressure on the flanges of the sleeve was remote, but it was one of the things which could be done at small expense and might help a little, one of the "straws"—so to speak—which a man will seek to grasp when in a difficult situation.

It should be noted that on one occasion in 1912 about two dozen of the flange bolts gave way (1¼-in. bolts, 3 in. from center to center), owing to the heavy pressure to which the pipe was subjected. The broken bolts were replaced with bolts of Norway iron, and there was no more trouble while the pipe was in use.

Mr. Clarke.

Except at one place, near the West Harbor line, no attempt was made to back-fill the pipe trenches after the pipes had been laid, and there a small quantity of gravel was placed around and over the pipe where it came near the surface of the river bed in its upward slope toward the bank.

In a few places the diver found it necessary to support a joint or two on blocking, in order to avoid too sharp an angle, but, as a rule, the pipes, after being laid on the trench bottom, were left to be covered by the silt carried by the first freshet, the depth of the pipe below the river bed being deemed an ample protection. It should be said, however, that vessels are not allowed to use that portion of the harbor as an anchorage ground, the river being also too narrow for such use by larger vessels.

Mr. Wiggin's inquiry regarding the shop methods used in the manufacture of the flexible joints has been answered by Mr. Phillips, as observed by him in the manufacture of the joints for the 30-in. pipe.

The inquiry of Mr. Spear regarding the accuracy of the final grade and alignment for the submerged pipes has already been touched on in reply to Mr. Wiggin.

Regarding the condition of the 28-in., cast-iron, ball-and-socket pipe taken up after 20 years' service, it may be said that the pipe itself, in the main, is in perfect condition, but slightly rusty. Before relaying, it will be necessary to turn down the balls slightly, merely to give them a fresh and uniform surface, no pitting of any depth having taken place.

Answering Mr. Hogan's request for information regarding the length of time required to make up and lay the flexible joints: Mr. Randlett notes that the length of time required for laying the underwater portion of the 30-in. line (an approximate length of 1 600 ft.) was only 9 days, or, say, 180 ft. per day; but this would involve the laying of only three of the flexible joints per day.

No attempt was made to smooth down the bed of the trench after the dredge had finished its work, the flexible joints being depended on to take care of such slight inequalities of surface as would be likely to occur.

The question raised by Mr. Brush as to the accuracy with which pipes can be laid under water, or "how much deviation in line and grade is permitted in submerged pipe construction", is of interest, but is also difficult to answer. In theory, the pipe should conform to the designated lines and grades; but, in practice, "it is different". In answer, therefore, the writer can only describe his experience with the several pipe-line crossings laid in Portland.

As has been stated, the 28-in., cast-iron, flexible-joint pipe, built in 1894, was laid in a trench excavated with a ladder-dredge which was supposed to excavate to a uniform depth below the surface of the water,

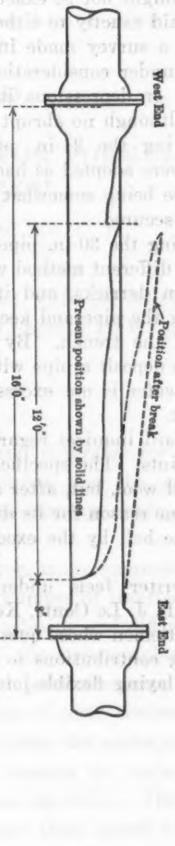
the ladder being adjusted according to the gauge showing the variation of water levels from the low-water line, which might occur from time to time. For laying the pipe, a timber cradle was constructed, one end of which rested on the pipe-laying barge and the other on the ground in the bottom of the excavated trench. This cradle was pulled from under the pipe as the pipe-laying barge was advanced, thus depositing the pipe in the trench along the line which the cradle followed, but it might or might not be exactly the alignment intended. That the pipe was not laid exactly to either line or grade as originally designed was shown by a survey made in 1902, when the question of lowering the pipe was under consideration. Soundings made at that time showed variations in alignment and grade amounting in places to as much as 2 ft., although no abrupt changes were noted. **Mr. Clarke.**

In laying the 28-in. pipe, built in [illegible], the same pipe-laying method were adopted as had been used 4 years before, but, on account of the pipe being somewhat lighter, it is presumed that a better alignment was secured.

In laying the 30-in. pipe in 1910, and in relaying the 24-in. pipe in 1913, a different method was adopted, the pipe being suspended from three boom derricks, and it was thus possible to line up the cables supporting the pipe and keep them in line until it was lowered to the bottom of the trench. By such means, the writer feels that it is possible to approach a pipe within a foot of the intended alignment, if the depth of water is not excessive. Any closer work than this he would not expect.

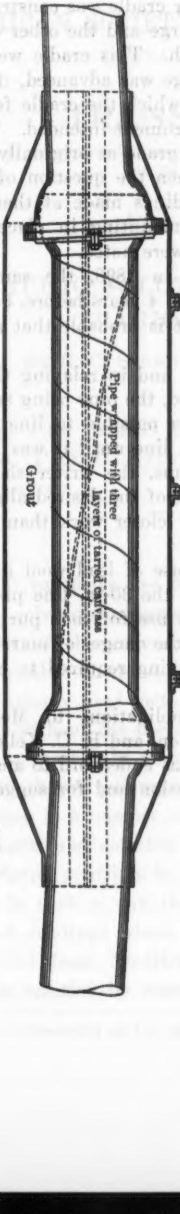
Mr. Ward inquires regarding the use of lead wool in calking the flexible joints. The specifications for the 30-in. pipe provided for the use of lead wool, but, after a trial, its use for this purpose was abandoned. One reason for its disuse was the danger of marring the turned face of the ball by the excessive calking required to consolidate the lead.

The writer feels under special obligations to Messrs. Clemens Herschel, F. J. Le Conte, Kenneth Allen, and [illegible], as well as to the gentlemen whose questions he has endeavored to answer, for their interesting contributions to the discussion and for suggested improvements in laying flexible-joint pipe.



PLAN OF BROKEN SECTION OF 28″ CAST-IRON SUBMERGED PIPE

FIG. 11.



REPAIR OF 28″ CAST-IRON SUBMERGED PIPE AT PORTLAND, ORE. SHOWING SPLIT SLEEVE IN PLACE

FIG. 12.

Mr. Clarke. the ladder being adjusted according to the gauge showing the variation of water levels from the low-water line, which might occur from time to time. For laying the pipe, a timber cradle was constructed, one end of which rested on the pipe-laying barge and the other on the ground in the bottom of the excavated trench. This cradle was pulled from under the pipe as the pipe-laying barge was advanced, thus depositing the pipe in the trench along the line which the cradle followed, but it might or might not be exactly the alignment intended. That this pipe was not laid exactly to either line or grade as originally designed was shown by a survey made in 1902, when the question of lowering the pipe was under consideration. Soundings made at that time showed variations or depressions in grade amounting in places to as much as 2 ft., although no abrupt changes were noted.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1333

NOMOGRAPHIC SOLUTIONS FOR FORMULAS OF VARIOUS TYPES*

By R. C. Strachan, M. Am. Soc. C. E.

With Discussion by Messrs. Allen Hazen, Carl B. Andrews, J. O. Eckersley, Paul C. Nugent, W. M. Eliot, F. W. Green, and R. C. Strachan.

Synopsis.

The present extensive use of charts and diagrams for shortening the labor of solving mathematical formulas may be accepted as sufficient proof of the general recognition among engineers of the value of graphical methods.

By the commonly used methods, however, many expressions cannot be represented without the use of a large number of intersecting lines, which cause the chart to be more or less confusing and difficult to read.

This paper shows the application of the nomographic principle of representation to a number of extensively used formulas, presenting illustrative charts and detailed descriptions of their construction.

The underlying principle is that of plotting the scales representing the variables in such a way that a line crossing the scales anywhere gives a set of readings which satisfy the equation. This principle has not received from American engineers that degree of attention to which it is entitled by reason of its great value and wide range

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* Presented at the meeting of February 3d, 1915.

of applicability, and the object in presenting the paper is to call attention to this somewhat neglected labor-saving device.

A nomographic representation, or, more concisely, a nomograph of a formula, may be defined as a plot or chart on which appear scales for the variables involved in the formula, their relative magnitudes and relative positions being such that corresponding values of the variables are found on a line crossing the scales.

By the expression "corresponding values" is meant a set of values which satisfy the equation represented by the nomograph; and the crossing line, therefore, is properly termed an isopleth.

The aim is to substitute the simplest possible mechanical operation for the labor of solving the formula by computation; and, with this in view, the nomograph should preferably be designed so that its isopleths are rectilinear.

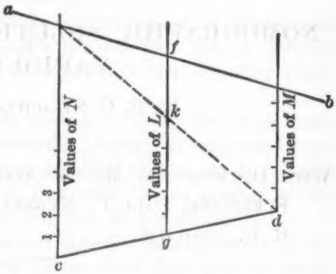


FIG. 1.

Take, as a simple illustration, three parallel scales of equal parts, starting from a common base line, *c d*, Fig. 1, and suppose the *L* line to be equidistant from the other two. Draw any straight line, *a b*, across the scales. Then $f\,g = f\,k + k\,g = \frac{1}{2}\,M + \frac{1}{2}\,N$.

The figure is a nomograph of the expression $L = \frac{1}{2}\,(M + N)$ and the line, *a b*, is an isopleth, its intersection showing by the scale readings a set of values which satisfy the equation.

If $g\,d = \frac{1}{3}\,c\,d$, $f\,g = \frac{1}{3}\,N + \frac{2}{3}\,M$, and the nomograph represents $L = \frac{N + 2\,M}{3}$, or $3\,L = N + 2\,M$, or $N = 3\,L - 2\,M$.

If the units on the *L* scale are made one-half as great as those on the other scales, the nomograph represents $L = \frac{2\,N + 4\,M}{3}$, or $3\,L = 2\,N + 4\,M$.

As another illustration, in Fig. 2, let $c\,d = d\,e$, and lay off the parallel scales for *P* and *Q* as shown. This is a nomograph of $P = \frac{1}{2}\,Q$, and any straight line drawn from *c* through the scales is an isopleth.

Evidently, changes in the positions and magnitudes of the scales may also be made in this diagram, with corresponding effects on the expression represented.

If in place of the numbers we lay off their logarithms, we may read products instead of sums; exponents may be affected by changing the distance between scales, and numerical coefficients may be introduced by varying longitudinally the position of one or more scales.

A straight-edge may be used to indicate the position of an isopleth, or, for greater ease in reading the scales, a strip of transparent celluloid having a line scratched on it, will be found very convenient.

The mathematical principles involved are very simple, and their application is easy when the right clue has been discovered. Many expressions which in their usual forms appear to be hopeless can be adapted to nomographic representation by a proper transformation; but the necessary change may not suggest itself until the equation has been studied somewhat thoroughly.

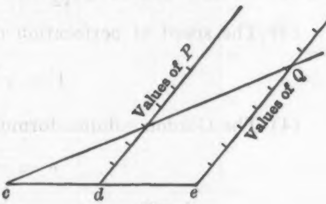


FIG. 2.

It will be found that, as a rule, scales of natural numbers are suitable for formulas in which the signs + and — appear; and logarithmic scales are adapted to those in which quantities affected by exponents must be multiplied or divided. In many cases, however, a formula in which there is an integral exponent may be transformed so as to permit the use of natural numbers for its nomographic scales.

Great ingenuity has been exhibited by various writers in constructing charts for formulas of much complexity: the work of Professor Maurice d'Ocagne is perhaps the most exhaustive, though both Soreau and Seco have shown that this field possesses astonishing possibilities.

Quite a large number of contributed articles on the subject have been published,* those of Professor John B. Peddle having been put into book form. Articles have also appeared by B. D. Dean, M. Am. Soc. C. E., and Mr. Richard Muller.†

At intervals, charts for complicated formulas have been published without explanation, except as to the method of using them; and though

* In the *American Machinist.*

† *Engineering News*, September 3d, 1908.

they are valuable to those who are satisfied as to their correctness, they are virtually empirical when used blindly.

Among earlier engineers, Messrs. Ganguillet and Kutter charted their well-known formula, and important additions to the chart were made afterward by Rudolph Hering, M. Am. Soc. C. E., who embodied his results in a paper* entitled "The Flow of Water in Small Channels."

A most interesting study of nomography was made by Mr. Rodolphe Soreau.† In addition to a new form of chart for Kutter's formula, Soreau has constructed nomographs for the solution of the following:

(1) The weir formula, in metric units,

$$Q = 95\,(H + 0.7)^{\frac{3}{2}};$$

(2) The volume of a conical frustum,

$$V = \frac{\pi\,h}{12}\,(D^2 + D\,d + d^2);$$

(3) The speed of perforation of a projectile,

$$V = q\,\frac{a^{0.75}}{p^{0.5}}\,t^{0.7};$$

(4) The Gordon column formula, expressed in metric units,

$$p = \frac{s}{1 + \dfrac{l^2}{n\,r^2}};$$

(5) The formula for the thickness of hollow cylinders,

$$m = \sqrt{\frac{R + p}{R - p}} - 1;$$

(6) The barometric formula for determining heights,

$$N = 18\,293\,(1 + 0.002837\ \cos.\ 2\,\lambda)\left[1 + \frac{2\,(T + t)}{1\,000}\right]\log.\ \frac{H}{h};$$

and many others.

The diagrams in common use almost invariably take the form of the locus of an equation referred to rectangular co-ordinates; and, in order to make them cover the required range, it is frequently necessary to use a great number of lines. This makes the diagram confusing and difficult to read, thus introducing chances for error. In case any of the factors are affected by exponents, the disadvantages of diagrams of this form are greatly increased.

* *Transactions*, Am. Soc. C. E., Vol. VIII, p. 1.

† The results are found in the Memoirs of the Society of Civil Engineers of France for 1901.

The immense superiority of the nomograph is due to the following facts: that it is easily read with precision on account of having few lines; that it saves much valuable time, providing in effect a perfect tabulation of all possible values within the limits of the chart; and that it enables one to see instantly the effect of a change, whether small or great, in any of the variables. Furthermore, the solutions are equally simple, whichever quantity in the formula is unknown.

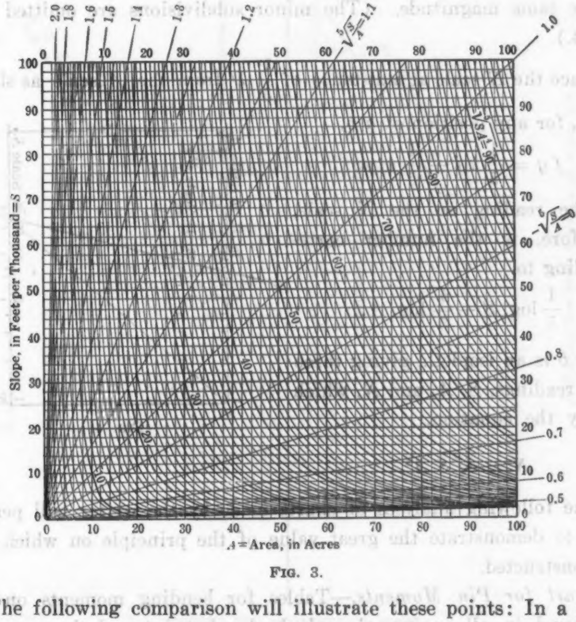


FIG. 3.

The following comparison will illustrate these points: In a recent issue of a technical periodical a correspondent gives the diagram, Fig. 3, for finding $\sqrt[5]{S A^4}$ in the McMath run-off formula.

He explains that, to construct this, it is necessary to compute $\sqrt[5]{\frac{S}{A}}$ and derive from this $\sqrt[5]{S A^4}$ for a number of cases sufficient to give all the points required for plotting the curves.

The finished diagram contains four sets of carefully plotted lines, and the intersections near the lower and left-hand edges are very indefinite.

A nomograph of this radical consists of three scales, and may be constructed with great ease and without computation, as follows:

Let $N = \sqrt[5]{S\,A^4}$.

Then $\log. N = \frac{1}{5}\log. S + \frac{4}{5}\log. A$.

Lay off three parallel scales of the logarithms of numbers from 1 to the limit desired (Fig. 4), making all three with logarithmic units of the same magnitude. (The minor subdivisions are omitted from Fig. 4.)

Place the N scale at a distance of $\frac{1}{5}\ D$ from the A scale as shown.

Then, for any position of $a\ b$,

$$f\,g = \frac{1}{5}\,k\,l + \frac{4}{5}\,m\,n.$$

The reading on the N scale, therefore, is the number corresponding to

$$\frac{1}{5}\log. S + \frac{4}{5}\log. A,$$

and $a\ b$ is an isopleth, giving three scale readings, S, A, and N, which satisfy the equation,

$$N = \sqrt[5]{S\,A^4}.$$

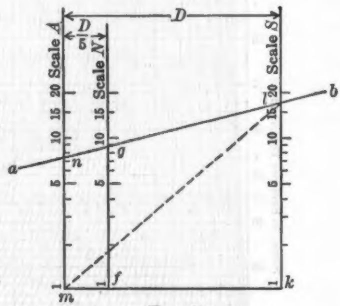


FIG. 4.

The following analyses of charts made by the writer will perhaps serve to demonstrate the great value of the principle on which they are constructed.

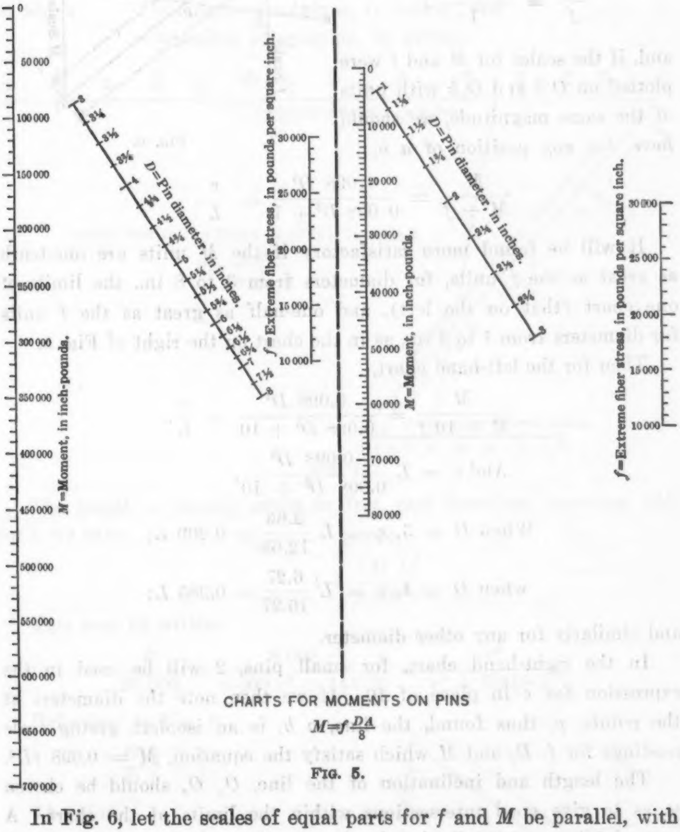
***Chart for Pin Moments.*—Tables for bending moments on pins are found in all engineers' pocketbooks, but interpolations are frequently necessary, particularly in reviewing a completed design. For this purpose a nomograph is especially useful, and a very neat one may be constructed with scales of natural numbers for moments and stresses. (Fig. 5.)**

Let M = moment, in inch-pounds;
D = diameter of the pin, in inches;
A = area of the pin, in square inches;
f = extreme fiber stress, in pounds per square inch.

Then $M = f\,\dfrac{D\,A}{8}$.

Or, by substituting $0.7854\,D^2$ for A,

$$M = 0.098\,f\,D^3.$$



CHARTS FOR MOMENTS ON PINS

$M = f\dfrac{DA}{8}$

FIG. 5.

In Fig. 6, let the scales of equal parts for f and M be parallel, with the zeros at O and O_1, as shown, and plotted at a convenient distance apart. Connect O_1 and O, and let any line, $a\ b$, intersect the three lines.

Then $$\frac{O\,l}{O_1\,k} = \frac{c}{e},$$

therefore,

$$\frac{O\,l}{O\,l + O_1\,k} = \frac{c}{L}.$$

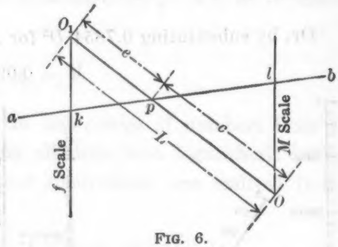


FIG. 6.

Also,

$$\frac{M}{f} = \frac{0.098\ D^3}{1},$$

and, if the scales for M and f were plotted on $O\ l$ and O_1k with units of the same magnitude, we should have, for any position of $a\ b$,

$$\frac{M}{M+f} = \frac{0.098\ D^3}{0.098\ D^3 + 1} = \frac{c}{L}.$$

It will be found more satisfactory if the M units are one-tenth as great as the f units, for diameters from 3 to 8 in., the limits of one chart (that on the left), and one-half as great as the f units for diameters from 1 to 3 in., as in the chart on the right of Fig. 5.

Then for the left-hand chart,

$$\frac{M}{M + 10\,f} = \frac{0.098\ D^3}{0.098\ D^3 + 10} = \frac{c}{L}.$$

$$\text{And } c = L\,\frac{0.098\ D^3}{0.098\ D^3 + 10}.$$

$$\text{When } D = 3,\ c = L\,\frac{2.65}{12.65} = 0.209\ L;$$

$$\text{when } D = 4,\ c = L\,\frac{6.27}{16.27} = 0.385\ L;$$

and similarly for any other diameter.

In the right-hand chart, for small pins, 2 will be used in the expression for c in place of 10. If we then note the diameters at the points, p, thus found, the line, $a\ b$, is an isopleth giving scale readings for f, D, and M which satisfy the equation, $M = 0.098\ fD^3$.

The length and inclination of the line, $O_1\ O$, should be chosen so as to give good intersections within the limits of the chart. A unit of 1 in. to 5 000 lb. for the f scale will be suitable for practical purposes.

Chart for Column Formula.—The Gordon column formula, when suitably transformed, may also be adapted to nomographic representation obtained by the relation of similar triangles.

The constants used in the chart, Fig. 7, are 20 000 and 8 000, or,

$$P = \frac{20\,000}{1 + \frac{l^2}{8\,000\,r^2}}$$

in which l = length of columns, in inches; and
r = radius of gyration, in inches.

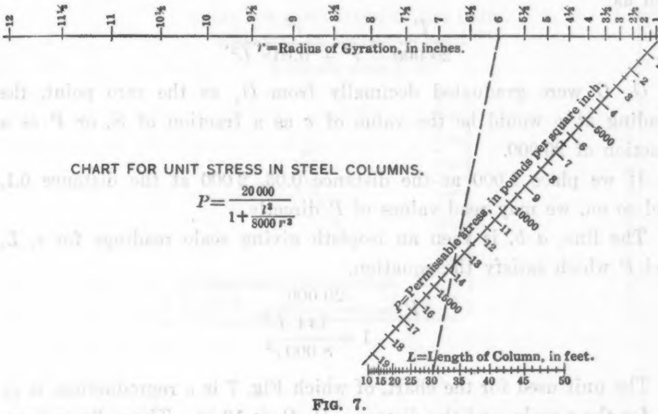


FIG. 7.

The length is usually given in feet, and therefore, inserting $12L$ for l, we have

$$P = \frac{20\,000}{1 + \frac{144\,L^2}{8\,000\,r^2}}.$$

This may be written

$$\frac{P}{20\,000} = \frac{8\,000\,r^2}{8\,000\,r^2 + 144\,L^2} = \frac{r^2}{r^2 + 0.018\,L^2}.$$

Referring to Fig. 8,

$$\frac{O_1\,k}{O\,l} = \frac{c}{d}.$$

Therefore,

$$\frac{O_1\,k}{O_1 k + O\,l} = \frac{c}{S}.$$

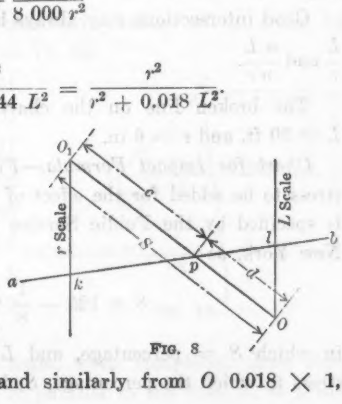


FIG. 8

If now we lay off from O_1 1, 4, 9, 16, etc., units, placing the numbers 1, 2, 3, 4, at these points, and similarly from O 0.018 $\times$ 1,

0.018×4, 0.018×9, 0.018×16 units, in other words, making the unit for the L scale 0.018 time the unit for the r scale, we shall have

$$\frac{r^2}{0.018\ L^2} = \frac{c}{d},$$

therefore,

$$\frac{r^2}{r^2 + 0.018\ L^2} = \frac{c}{S},$$

and as

$$\frac{P}{20\ 000} = \frac{r^2}{r^2 + 0.018\ L^2},$$

if $O_1\ O$ were graduated decimally from O_1 as the zero point, the reading at p would be the value of c as a fraction of S, or P as a fraction of 20 000.

If we place 1 000 at the distance 0.05, 2 000 at the distance 0.1, and so on, we may read values of P directly.

The line, $a\ b$, is then an isopleth giving scale readings for r, L, and P which satisfy the equation,

$$P = \frac{20\ 000}{1 + \dfrac{144\ L^2}{8\ 000\ r^2}}.$$

The unit used for the chart, of which Fig. 7 is a reproduction, is $\frac{1}{10}$ in. for the r scale, and the distance, $O_1\ O$, is 10 in. These dimensions, of course, must be chosen with regard to the degree of precision desired in the scale readings. When constructed as stated, the value of P may be read to 100 lb.

Good intersections may always be obtained, since P is the same for $\frac{L}{r}$ and $\frac{n\ L}{n\ r}$.

The broken line on the chart shows the solution for the case $L = 30$ ft. and $r = 6$ in.

Chart for Impact Formula.—Fig. 9.—The percentage of live-load stress to be added for the effect of impact in designing bridge trusses is specified by the Public Service Commission of the First District, New York, as

$$S = 125 - \frac{1}{8}\sqrt{2\ 000\ L - L^2}$$

in which S = percentage, and L = length of load producing the stress to which the percentage, S, is to be added.

If we represent $2\,000\ L - L^2$ by q, this may be written

$$125 - S = \frac{1}{8}\sqrt{q}.$$

Also, $$\log.\ (125 - S) = \frac{1}{2}\log.\ q - \log.\ 8.$$

The series of values of q, taken with suitable intervals in the value of L, may be easily deduced, because, for consecutive values

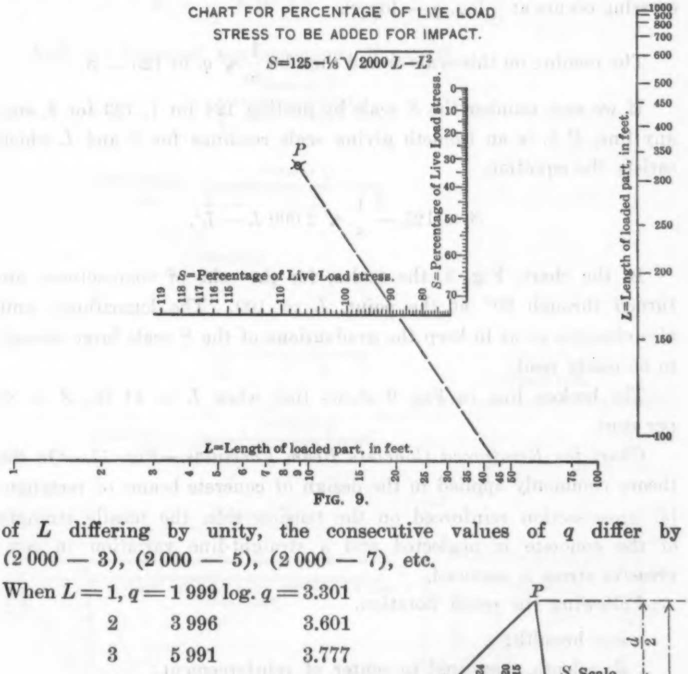


FIG. 9.

of L differing by unity, the consecutive values of q differ by $(2\,000 - 3)$, $(2\,000 - 5)$, $(2\,000 - 7)$, etc.

When $L = 1$,	$q = 1\,999$	$\log.\ q = 3.301$
2	3 996	3.601
3	5 991	3.777
4	7 984	3.902
5	9 975	3.999

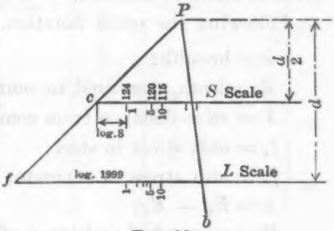


FIG. 10.

Referring to Fig. 10, lay off the logarithms of q from the point, f, marking 1 opposite log. 1 999, 2 opposite log. 3 996, etc.

From the extremities of this scale, when completed, draw lines to a point, P, and place the line, S, parallel to the line, L, at a dis-

tance of $\frac{1}{2}d$ from it. If a logarithmic scale of the same unit magnitude as the scale, L, were placed on the line, S, beginning at c, any straight line from P through the L scale would give the number corresponding to $\frac{1}{2}$ log. q at the point of crossing the line, S; but, by beginning at a distance of $0.908 =$ log. 8 from c, the point of crossing occurs at $\frac{1}{2}$ log. q — log. 8.

The reading on this scale would then be $\frac{1}{8}\sqrt{q}$, or $125 - S$.

If we now number the S scale by putting 124 for 1, 123 for 2, etc., any line, $P\ b$, is an isopleth giving scale readings for S and L which satisfy the equation,

$$S = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2}.$$

In the chart, Fig. 9, the scales, for the sake of compactness, are turned through 90° at the point, $L = 100$. The logarithmic unit also changes, so as to keep the graduations of the S scale large enough to be easily read.

The broken line in Fig. 9 shows that when $L = 44$ ft., $S = 88$ per cent.

Chart for Reinforced Concrete Beam Formulas.—Fig. 11.—On the theory commonly applied in the design of concrete beams of rectangular cross-section reinforced on the tension side, the tensile strength of the concrete is neglected and a straight-line variation in compressive stress is assumed.

Following the usual notation,

$b =$ breadth;
$d =$ depth, measured to center of reinforcement;
$X = xd =$ distance from compression side to neutral axis;
$f_s =$ unit stress in steel;
$f_c =$ unit stress in concrete;
$n = E_s \div E_c$;
$M =$ moment of resistance of beam;
$p =$ percentage of reinforcement;
$a =$ area of reinforcement per unit width $= p\ d$.

Then $\quad X = x\,d = d\left(\sqrt{2\,p\,n + p^2\,n^2} - p\,n\right),$

$$a = p\,d = \frac{d}{2\left(\dfrac{f_s}{f_c} + \dfrac{f_s^{\,2}}{n\,f_c^{\,2}}\right)},$$

$$M_s = f_s\,p\,d^2\left(1 - \frac{x}{3}\right),$$

$$M_c = \frac{1}{2}\,f_c\,d^2\left(x - \frac{x^2}{3}\right).$$

And, for balanced reinforcement, $M_s = M_c$.

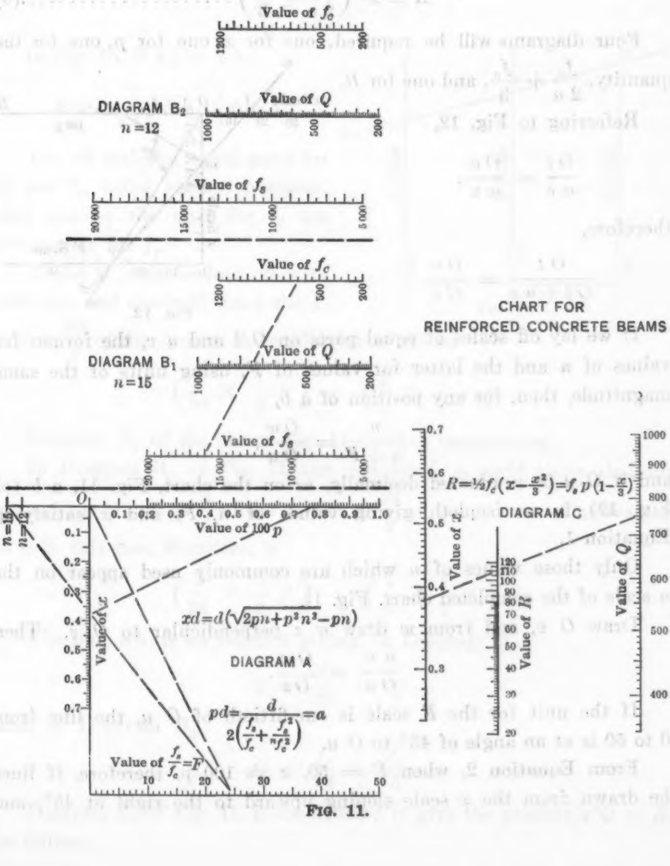


FIG. 11.

These expressions may be transformed so as to adapt them to nomographic representation.

If
$$\frac{f_s}{f_c} = F,$$
and the moment of resistance $R\, b\, d^2$, or $R\, d^2$ per unit width,

then
$$x = \frac{n}{n+F} \qquad (1)$$
$$p = \frac{x}{2F} \qquad (2)$$
$$R = x^2\left(\frac{f_s}{2n} + \frac{f_c}{3}\right) \qquad (3)$$

Four diagrams will be required, one for x, one for p, one for the quantity, $\frac{f_s}{2n} + \frac{f_c}{3}$, and one for R.

Referring to Fig. 12,
$$\frac{O\,t}{u\,v} = \frac{O\,w}{w\,u},$$
therefore,
$$\frac{O\,t}{O\,t + u\,v} = \frac{O\,w}{O\,u}.$$

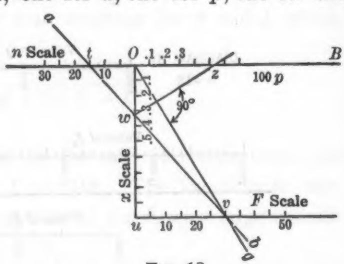


FIG. 12.

If we lay off scales of equal parts on $O\,t$ and $u\,v$, the former for values of n and the latter for values of F, using units of the same magnitude, then, for any position of $a\,b$,
$$\frac{n}{n+F} = \frac{O\,w}{O\,u},$$
and if $O\,u$ be graduated decimally, as on the chart, Fig. 11, $a\,b$ (of Fig. 12) is an isopleth giving values of n, F, and x satisfying Equation 1.

Only those values of n which are commonly used appear on the n scale of the completed chart, Fig. 11.

Draw $O\,v$, and from w draw $w\,z$ perpendicular to $O\,v$. Then
$$\frac{u\,v}{O\,u} = \frac{O\,w}{O\,z}.$$

If the unit for the F scale is one-fiftieth of $O\,u$, the line from 0 to 50 is at an angle of 45° to $O\,u$.

From Equation 2, when $F = 50$, $x = 100\,p$, therefore, if lines be drawn from the x scale sloping upward to the right at 45°, and

if the intersections with $O\ B$ are marked with the same figures as the points on the x scale from which they start, we shall have the scale of $100\ p$ on $O\ B$.

A line, $O\ g$, through the F scale and a line perpendicular to $O\ g$ through w and $O\ B$ will then give values of F, x, and p, satisfying Equation 2. This completes Diagram A of the chart, Fig. 11.

To obtain R it is necessary first to obtain $\frac{f_s}{2\ n} + \frac{f_c}{3}$. Denoting this quantity by Q, when $n = 15$,

$$Q = \frac{f_s}{30} + \frac{f_c}{3}.$$

In Fig. 13, if $k\,g = k\,h$,

$$k\,m = \frac{1}{2}\,g\,n + \frac{1}{2}\,h\,p.$$

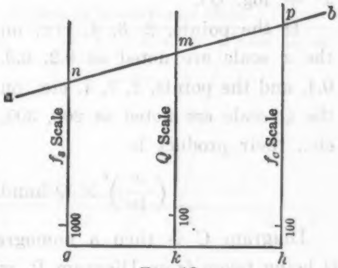


FIG. 13.

Lay off scales of equal parts for f_s and f_c, using natural numbers, and making the unit for f_c ten times that for f_s.

If the Q scale be laid off with unit one and one-half times the f_c unit or $\frac{30}{2}$ of the f_s unit, the reading at m will be

$$\left(\frac{2}{30} \times \frac{1}{2} f_s\right) + \left(\frac{2}{3} \times \frac{1}{2} f_c\right).$$

Diagram B_1 of the chart, Fig. 11, is thus constructed.

In Diagram B_2, of Fig. 11, the unit for f_c is eight times that for f_s, and the Q unit is $\frac{3}{2}$ of the f_c unit or $\frac{24}{2}$ of the f_s unit. The reading at m in this case, therefore, is

$$\left(\frac{2}{24} \times \frac{1}{2} f_s\right) + \left(\frac{2}{3} \times \frac{1}{2} f_c\right).$$

The line, $a\,b$, is an isopleth giving, in Diagram B_1,

$$Q = \frac{f_s}{30} + \frac{f_c}{3},$$

and, in Diagram B_2,

$$Q = \frac{f_s}{24} + \frac{f_c}{3}.$$

Diagram C, of Fig. 11, is constructed to give the product $x^2 Q = R$, as follows:

Place the R line as shown in Fig. 14, making $k\,g = \frac{1}{3}\,g\,h$. Lay off the logarithms of x and Q on their respective lines, using the same unit. Then, for any position of $a\,b$,

$$k\,m = \frac{2}{3}\log.\,x + \frac{1}{3}\log.\,Q = \frac{1}{3}\,(\log.\,x^2 + \log.\,Q).$$

The R scale is laid off with unit one-third of that used for x and Q, making the reading at m the number corresponding to (log. x^2 + log. Q).

If the points, 2, 3, 4, etc., on the x scale are noted as 0.2, 0.3, 0.4, and the points, 2, 3, 4, etc., on the Q scale are noted as 200, 300, etc., their product is

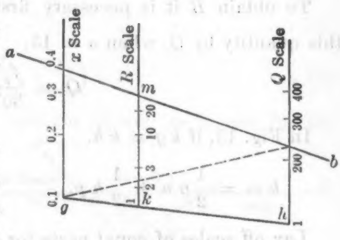


FIG. 14.

$$\left(\frac{x}{10}\right)^2 \times Q \text{ hundreds} = x^2\,Q \text{ units.}$$

Diagram C is then a nomograph for the expression, $R = x^2Q$, Q being taken from Diagram B_1 when $n = 15$, and from Diagram B_2 when $n = 12$, f_c and f_s being in pounds per square inch, b and d in inches, and $R\,b\,d^2$ in inch-pounds.

To illustrate the use of the chart, Fig. 11, assume $f_s = 16\,000$, $f_c = 650$, $n = 15$, and $b = 12$ in.

Moment of external forces = 517 000 in-lb.

On Diagram A connect the point n 15 with F 24.6 and read $x = 0.38$.

Connect the origin, O, with F 24.6, and, on a perpendicular to this line, drawn from 0.38 on the x scale, read $100\,p = 0.77$, or $p = \frac{77}{100}$ of 1 per cent.

On Diagram B_1 connect f_s 16 000 with f_c 650, and read $Q = 750$.

On Diagram C connect x 0.38 with Q 750, and read $R = 109$.

$$\text{Then } 517\,000 = 109 \times 12\,d^2,$$
$$\text{therefore, } d = 20 \text{ in.}$$

Allowing 2 in. of concrete below the center of reinforcement, the required beam would then be 12 by 22 in., and the steel area would be 0.77% of 240 sq. in. = 1.85 sq. in.

Charts for Williams-Hazen Formula.—Many hydraulic formulas contain fractional exponents, and are cumbersome on account of the time required for their solution. This difficulty is partly overcome by tabulating the results for a series of cases. Interpolations in such tables, however, are not invariably simple, and resort is had to many expedients to simplify the work. Special "hydraulic slide-rules" have

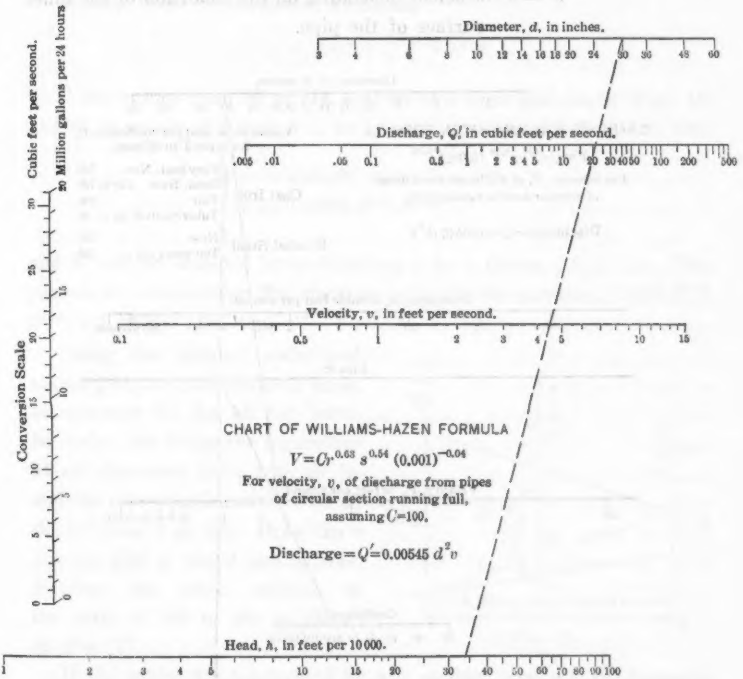


FIG. 15.

been devised, which give, with sufficient precision, the results of the formula for which they are designed, but naturally such special rules can be of no generaly utility.

The charts for the Williams-Hazen formula, Figs. 15 and 16, are given as examples of the beauty of the nomograph in its application to the solution of an exponential equation.

As usually stated, this formula is

$$V = Cr^{0.63}S^{0.54}0.001^{-0.04},$$

in which V = velocity, in feet per second;
r = mean hydraulic radius;
S = slope or ratio of rise to length;
C = a coefficient depending on the condition of the inner surface of the pipe.

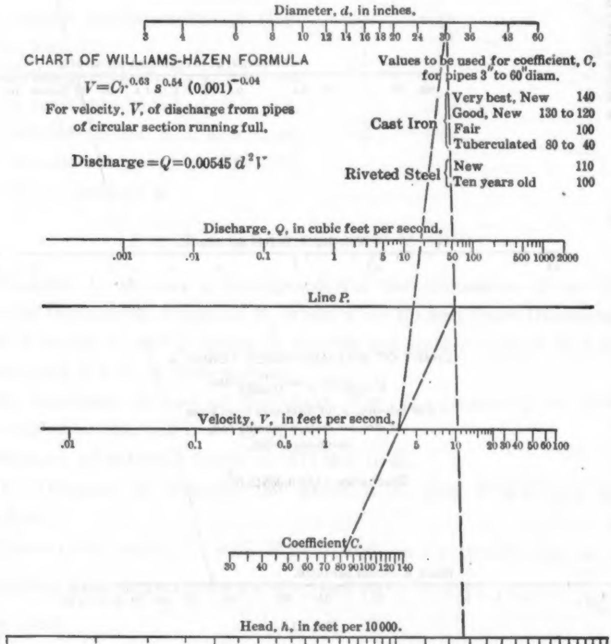


FIG. 16.

In constructing a nomograph of this formula, applicable to pipes of circular section running full, it will be convenient first to transform it so as to express r in terms of the diameter, and s as feet of head in 10 000 ft.

Letting d = diameter of pipe, in inches;
and h = feet of head in 10 000 ft.,

$$\text{then } r = \frac{D \text{ (feet)}}{4} = \frac{d \text{ (inches)}}{48},$$

$$\text{and } S = \frac{h}{10\,000}.$$

Making these substitutions, we obtain

$$V = 0.000796\ C\ d^{0.63}\ h^{0.54}.$$

C has values from 30 to 140, and we will limit the charts, Figs. 15 and 16, to values of d from 3 to 60 in. and to values of h from 1 to 100.

If v = velocity when $C = 100$,

$$v = 0.0796\ d^{0.63}\ h^{0.54},$$

and V will be obtained by multiplying v by a factor, 0.3 to 1.4. The process of constructing the chart so as to give the product, $0.0796\ d^{0.63}\ h^{0.54}$, will be the first step.

Using the decimal scale, and taking ½ in., or 1 on the 20th scale, to represent 0.1, lay off two parallel scales, one being the logarithms of all diameters from 1 to 60 in. and the other the logarithms of all the h's from 1 to 100. Draw the v line parallel to the d and h lines, dividing the space between in the ratio of 63 to 54, as shown in Fig. 17.

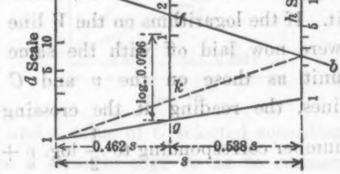


FIG. 17.

If the scales are intersected by any straight line, $a\ b$, the distance,

$$f\,g = f\,k + k\,g = \frac{63}{117} \log. d + \frac{54}{117} \log. h.$$

If the v's were laid off by the scale of 20ths beginning at the point, g, the reading at f would give the product, $d^{\frac{63}{117}}\ h^{\frac{54}{117}}$.

$\frac{63}{117} = 0.855 \times \frac{63}{100}$, hence if the logarithms of the v's are laid off with a unit of (½ in. × 0.855) to 0.1, the reading at f would give the

product, $d^{0.63}\ h^{0.54}$. This may be accomplished by using for the v's the scale of 20ths after multiplying the logarithms by 0.855, thus :

$$\log.\ 2 = 0.301 \times 0.855 = 0.257$$
$$\log.\ 3 = 0.477 \times 0.855 = 0.408$$
$$\log.\ 4 = 0.602 \times 0.855 = 0.515,$$

or by any other convenient method of reducing the unit magnitude.

The logarithm of $0.0796 = \bar{2}.901 = -\ 1.099$, and by placing the starting point of the v's above g a distance $1.099 \times 0.855 = 0.940$ on the scale of 20ths, thus adding $-\ 1.099$, the logarithm of 0.0796, the reading on the v line is $0.0796\ d^{0.63}\ h^{0.54}$.

To introduce the factor, C, draw the C line parallel to the others at any suitable place between the v line and the h line. Draw the V line midway between the v line and the C line, as in Fig. 18.

From the point, 1, on the v scale, draw a line across the v, V, and C lines at any convenient angle, in order to locate the "1" points of the scales about to be plotted. Using the same unit as for v, lay off on the C line the logarithms of 1.1, 1.2, 1.3, and 1.4 above 1 and the logarithms of 0.9, 0.8, etc., below it. If the logarithms on the V line were now laid off with the same unit as those on the v and C lines, the reading at the crossing of any line, $a\ b$, would be the number corresponding to $\frac{1}{2}\log.\ v + \frac{1}{2}\log.\ C$.

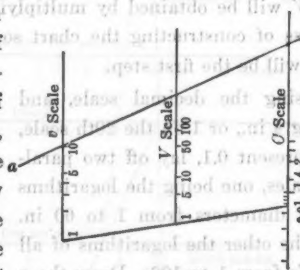


FIG. 18.

Therefore we make the V scale one-half as large as the other two, that is, with a unit of ($\frac{1}{2}$ in. $\times$ 0.855), which will give the product, $v\ C$, as required.

It is apparent that no divisions are required on the v line after the starting points for the V and C scales are drawn. Therefore it is left blank in the completed chart, Fig. 16, and marked "Line P" for reference.

In order to make the chart of greater use, it will be well to introduce another scale giving Q, the quantity discharged.

If $Q =$ discharge, in cubic feet per second;
$d =$ diameter, in inches;
$V =$ velocity, in feet per second;
$Q = 0.00545 d^2 V$.

The d scale was plotted with a unit of $\frac{1}{2}$ in. to 0.1. Therefore it will serve for log. (d^2) with a unit of $\frac{1}{4}$ in. to 0.1. The V scale is made with a unit of ($\frac{1}{4}$ in. $\times$ 0.855), as shown on Fig. 19.

If we place the Q line between the d and V lines, as shown, the distance, $f\,g = n \times \log.\ (d^2) + (1 - n) \times 0.855 \log.\ V$, and if $n = (1 - n) \times 0.855$,

$$n = 0.462$$

then $\qquad f\,g = 0.462 \log.\ (d^2) + 0.462 \log.\ V.$

If the Q scale were now laid off from g with a unit of ($\frac{1}{4}$ in. $\times$ 0.462), the reading at f would be the number corresponding to log. $d^2 + \log.\ V$.

Log. $0.00545 = \bar{3}.7364 = -2.2636$, and if we place the "1" of the Q scale above g a distance, $2.2636 \times 0.462 = 1.045$ on the 40th scale ($\frac{1}{4}$ in. representing 0.1), thus adding log. 0.00545, the reading at the intersection of $a\ b$ will give $0.00545\ d^2\ V$, as required.

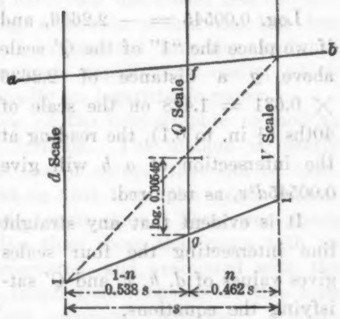


FIG. 19.

Assuming d and h to be known, and a value of C selected according to the condition of the pipe, place a straight-edge so as to connect d and h, and mark its intersection on "Line P". Similarly, connect the point just marked with the given C and at the intersection on the V line read the value of V. Then connect this V with the given d, and at the intersection on the Q line read Q in cubic feet per second.

It is evident that, by a suitable modification of the process described, either of the four quantities, d, h, V, or C, may be obtained when the other three are given.

As C is usually assumed at a round figure, multiplication or division by it is so simple that some might prefer a chart based on

the value 100 for this coefficient. The chart, Fig. 15, is constructed in this way, as follows:

Lay off the d, h, and v scales as described. The v scale will have its subdivisions completed and will occupy the position of "Line P" in the chart, Fig. 16.

The subdivisions on the d line represent log. (d^2) with a unit of $\frac{1}{4}$ in., and those on the v line represent log. v with a unit of ($\frac{1}{2}$ in. $\times$ 0.855), equivalent to $\frac{1}{4}$ in. $\times$ 1.71.

If we place the Q' line as shown in Fig. 20, the distance,

$$f\,g = m \text{ log. } (d^2) + (1 - m) \times 1.71 \text{ log. } v;$$

$$\text{and if } m = (1 - m) \times 1.71,\ m = 0.631.$$

Then $$f\,g = 0.631 \text{ log. } (d^2) + 0.631 \text{ log. } v.$$

If the Q' scale were laid off from g with a unit of ($\frac{1}{4}$ in. $\times$ 0.631), the reading at f would be the number corresponding to log. (d^2) + log. v.

Log. 0.00545 = $-$ 2.2636, and if we place the "1" of the Q' scale above g a distance of 2.2636 $\times$ 0.631 = 1.428 on the scale of 40ths ($\frac{1}{4}$ in. to 0.1), the reading at the intersection of a b will give $0.00545d^2v$, as required.

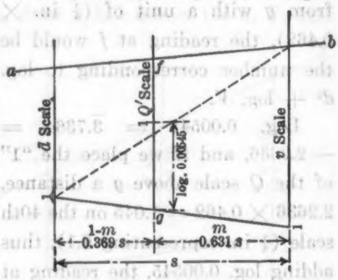


FIG. 20.

It is evident that any straight line intersecting the four scales gives values of d, h, v, and Q' satisfying the equations,

$$v = 100 \times 0.000796\ d^{0.63}\ h^{0.54},$$

$$\text{and } Q' = 0.00545d^2v.$$

By means of the "conversion scale", cubic feet per second may be read off at once, in millions of gallons per day.

The broken lines drawn on the charts, Figs. 15 and 16, show that when d = 30 in., h = 33 ft., and C = 82, the formula gives V = 3.7 and Q = 18 cu. ft. per sec. Also when d = 30 in. and h = 33 ft., v = 4.5 and Q' = 22 cu. ft. per sec.

DISCUSSION

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—This paper is important, as it calls attention to a method of easy and rapid graphical solution of many formulas, which is capable of wide application, especially where only three variables are involved. In the common hydraulic formulas there are four variables, namely, slope, radius, coefficient, and velocity (or quantity). Although the method can be extended to cover the four variables, much of its ease of application is lost when the fourth is added. Mr. Hazen.

It is certainly true that much time is spent by some people in making computations for the production of diagrams which have little practical use. On the other hand, diagrams which are useful, can frequently be made. The diagram cited as a horrible example illustrates this. As presented, it was hard to make, and is of little use. With logarithmic paper, a parallel rule, and a few primary points, a more useful diagram could have been made with less effort.

The introduction of the log. log. slide-rule within the last few years has facilitated the solution of exponential formulas, and has permitted many calculations to be made rapidly and easily, which formerly could best be solved graphically on logarithmic paper.

The author shows the application of the nomographic method to the Williams-Hazen hydraulic-flow formula. The method is clearly well adapted to this service, and is one of several that can be used.

Before this formula had been put in final form, the writer experimented with slide-rules of home-made construction. These were made by constructing slides a little smaller than those that came with an ordinary Manheim slide-rule, paper-covered, and graduated with the desired scale. In this way it was found that formulas of this type could be solved rapidly with sufficient accuracy. Afterward, a slide-rule was designed for the solution of this formula, a maker found, and several hundred such rules have been sold.

Engineers, like other people, have strongly fixed habits. Some engineers can best solve their problems on a slide-rule, some by the use of diagrams, and others by printed tables. For the solution of the Williams-Hazen hydraulic formula, at least five procedures may be used: (1) the hydraulic slide-rule, made by Ledder and Probst, of Boston; (2) "Hydraulic Tables", published by John Wiley and Sons; (3) diagrams on logarithmic paper, of which there are a number of very excellent ones, such as those published in some of the engineering papers; (4) the procedure suggested by the author; and (5) direct solution with a log. log. slide-rule.

The writer believes that those who have used the slide-rule sufficiently to become accustomed to it would not be willing to adopt any

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Mr. Hazen.

of the other methods. Getting accustomed to the slide-rule is not difficult, but many engineers are reluctant even to make this effort, and it is interesting to note that eight times as many copies of the "Hydraulic Tables" have been sold as of the slide-rules. Also, notwithstanding the existence of both the slide-rule and the tables, several members of the Society have thought it worth their while to make special diagrams on logarithmic paper for the solution of the formula. The nomographic method will no doubt appeal to others, and will be useful to them.

By calling attention to an excellent method of solving formulas of this type rapidly, the author has rendered a substantial service to the Profession.

Mr. Andrews.

CARL B. ANDREWS,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Strachan remarks in his synopsis that the principle of the isopleth has not received from American engineers that degree of attention to which it is entitled. This statement is undoubtedly true, for the alignment diagram has all the advantages over diagrams platted by rectangular co-ordinates, which Mr. Strachan claims, and more.

Some years ago the writer read part of M. d'Ocagne's treatise on Nomography, and became interested to the extent of drawing a number of alignment diagrams, and writing a paper on the subject, which was read before a local engineering society. Of the diagrams produced, however, only two have ever been used enough to justify the time spent in making them, these two being intended for the reduction of stadia readings to vertical and horizontal distances. It seems to the writer that many of the mathematical operations which occur in civil engineering work require a higher degree of accuracy than is obtainable from diagrams of any kind, and that the diagrams find their proper use in checking computations and in making quick approximate calculations.

A class-room remark of the late Professor Thomas Gray, which strongly impressed the writer, was to the effect that there was a disadvantage in the use of reckoning machines, diagrams, multiplication tables, and other such mechanical aids in computation, in that the computer was likely to get into the habit of writing down the results without noting the intermediate steps, which made it very hard to trace errors resulting from an accidental wrong entry or wrong reading. In the preparation of the Smithsonian Physical Tables, Dr. Gray said that much computation had been required, but that comparatively few errors had been found in the work after its publication, because the computations had been carried out in full in note-books, where they were readily checked.

In offices where there is a great deal of computation, of a kind in which diagrams may be used to advantage, alignment diagrams, as

* Honolulu, Hawaii.

described by Mr. Strachan, are undoubtedly much more convenient and useful than those platted by rectangular co-ordinates. Mr. Andrews.

J. O. Eckersley,* M. Am. Soc. C. E.—The speaker is of the opinion that this paper has not made it entirely clear that these so-called nomographs are nothing more or less than graphic tables which have been prepared by giving the variables in an equation successive integral values, for a considerable range, and plotting the function (with proper units) on a system of co-planar lines, approximating the intervening values of the variable by eye, instead of tabulating them in the usual manner. For example, let $4x + 5y = 60$. Tabulating, we have Mr. Eckersley.

x.	y.	Constant.	y.	x.
0	12	60	0	15
1	11.2	....	1	13.75
2	10.4	....	2	12.50
3	9.6	....	3	11.25
4	8.8	....	4	10.

The nomograph is Fig. 21, and the diagram, using Cartesian co-ordinates, is Fig. 22.

Fig. 21 (parallel co-ordinates) and Fig. 22 (rectangular co-ordinates) illustrate the "principle of duality", that is, a relation of lines and

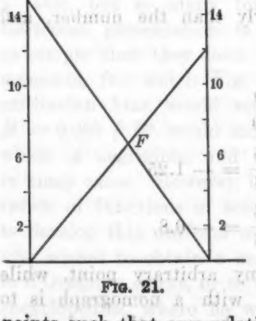

Fig. 21.

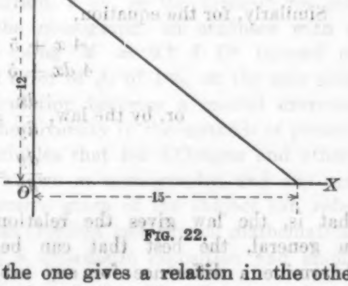

Fig. 22.

points such that any relation in the one gives a relation in the other by interchanging the words "line" and "point". For example, in Fig. 21 two lines determine a point, F; and in Fig. 22 two points (0, 12) and (15, 0) determine a line.

* New York City.

Mr. Eckersley.

In 1884, or about 30 years ago, this particular system of tabulation was introduced, and since then scores of writers have contributed to it. In America some use has been made of these diagrams by mechanical and electrical engineers, but they were known by other names, such as "Graphical Arithmetic", "Graphic Charts", etc. The word "nomograph", by itself considered, is a misnomer. Etymologically, it means "a written law". As a matter of fact, nomographs do not give the laws governing the function any more than a table of logarithms gives the law governing logarithms. Law, in a mathematical sense, is defined as: The rule or formula by which certain functions vary, or according to which certain changes take place, for example:

$$x = e^y$$

$$\text{or } y = \log. x = \text{the logarithmic curve.}$$

Then $$dy = \frac{1}{x}\, dx \text{ is the law,}$$

$$\frac{dy}{dx} = \frac{1}{x} = \tan. \phi.$$

$$\text{when } x = 1, \tan. \phi = 1\text{; that is, } \phi = 45^\circ$$
$$\text{when } x > 1, \tan. \phi < 1\text{; that is, } \phi < 45^\circ$$
$$\text{when } x < 1, \tan. \phi > 1\text{; that is, } \phi > 45^\circ$$

which proves that the curve (Fig. 23) lies entirely below the tangent at $x = 1$. From $dy = \frac{1}{x}\, dx$ it is seen that when the number, x, is greater than 1, the logarithm changes more slowly than the number, and conversely.

Similarly, for the equation,

$$4\,x + 5\,y = 60$$
$$4\,dx + 5\,dy = 0$$
$$\text{or, by the law, } \frac{dx}{dy} = -\frac{5}{4} = -1.25$$
$$\frac{dy}{dx} = -\frac{4}{5} = -0.8$$

that is, the law gives the relation at any arbitrary point, while in general, the best that can be done with a nomograph is to determine a difference for any arbitrary values on the scales, but from the data obtained there is no assurance of what the difference would be on any other part of the scales.

As these tables depend on measurements, more than on anything else, it would seem that "metrical tables" is a far more suggestive

Mr. Eckersley.

title; and that the well-known mathematical term, "connector" would be much more appropriate than the word "isopleth". The terms "metrical tables" and "connector" suggest relevant and important ideas, as names should always do.

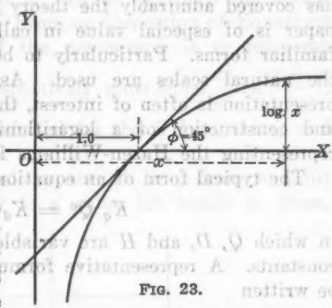

FIG. 23.

Passing to their practical value, it is noted that the method is analytical, but the conceptions and representations are purely metrical, and not to be confused with, or related to, grapho-statics, the difference being that the latter deduces from graphs results of the same form, by means of lines derived systematically from certain fundamental notions, for example, the funicular polygon, the reciprocal figures of Cremona, Maxwell, etc. Enough has been shown by the author to establish the claim that these tables are worthy of study, but, unless they have been studied and one has acquired the ability to select the proper scales and transform the equations into such forms as permit of this tabular plotting, it will be very much like one who reads poetry and has to spell each word; in other words, "one will not go far"; and it is inability to do those things which explains why these diagrams are not more popular.

When an engineer uses a particular formula (involving several variables) very frequently, it might be of advantage to him to prepare a table, but so many formulas depend on two variables that the Cartesian presentation is entirely satisfactory. Indeed, many are so simple that they need no diagram, *e. g.,* in the formula for pin moments, for which Fig. 5 is the nomograph, an engineer with a utilitarian bias would see that using $M = 0.1\ f\ D^3$ instead of $M = 0.098\ f\ D^3$ would induce an error of $\frac{2}{10}$ of 1%, on the safe side, which is negligible, and the calculation becomes a mental exercise, in many cases. However, it was the diversity in the methods of presentation of functions of several variables that led d'Ocagne and others to develop this uniform method, known as nomography, and any one who wishes to obtain a comprehensive grasp of the subject can refer to d'Ocagne's works, or to the great German and French mathematical encyclopedias, where he will find a systematic exposition and ample references to original memoirs.

The author is to be congratulated for this able presentation. The proofs are concise, and no doubt his paper has brought to the attention of many a very useful and convenient way of tabulating a function of several variables.

Mr. Nugent.

PAUL C. NUGENT,* M. AM. SOC. C. E. (by letter).—Mr. Strachan has covered admirably the theory of the nomographic diagram. His paper is of especial value in calling attention to some of the less familiar forms. Particularly to be noted are the diagrams in which the natural scales are used. As a somewhat different method of presentation is often of interest, the following treatment of the theory and construction of a logarithmic nomograph, similar to the one representing the Hazen-Williams formula, is here submitted.

The typical form of an equation of this kind is

$$K_q\, Q^a = K_d\, D^b \times K_h\, H^c \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which Q, D, and H are variables, and K_q, K_d, K_h, a, b, and c, are constants. A representative formula is that of Flamant, which may be written

$$Q = 0.001361\, D^{\frac{19}{7}}\, H^{\frac{4}{7}} \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In Equation (2) Q is in cubic feet per second; D is the diameter of the pipe, in inches; and H is the drop, in feet, of the hydraulic gradient per 1 000 ft. of pipe line.

Referring to Fig. 24, on the vertical line, H, are supposed to be laid off, from the point H downward, the logarithmic values of $K_h\, H^c$. It is next necessary to assume the distances, x and y. These, for convenience, may be equal. Having assumed them, draw two vertical lines, D and Q, as shown. Assuming any convenient point, Q, on the Q line, draw a straight line, RS, through this point and H. Q is the beginning or zero point of the Q line, and D, the intersection of RS with the intermediate vertical previously drawn, is the corresponding point on the D line.

To discuss first the case where the Q, D, and H factors, including their constant exponents and coefficients, are each not less than unity, suppose any straight-edge, $L_1\, M_1$, laid across the diagram as shown in Fig. 24. It is evident that the Q and D quantities must increase upward, and the H quantities downward, from what may be termed the "zero line", RS. Consequently, in the assumed case (all factors not less than unity and therefore all logarithms plus) $L_1\, M_1$ will intersect RS to the right of the D line, and will cut the three verticals in the points, Q_1, D_1, and H_1.

From the figure,

$$\frac{HH_1 + QQ_1}{x + y} = \frac{HH_1 + DD_1}{y}, \text{ from which}$$

$$QQ_1 = \frac{x}{y}\, HH_1 + \frac{(x+y)}{y}\, DD_1 \ldots\ldots\ldots\ldots (3)$$

Now, suppose that the scale to which the Q quantities are laid off is n_h and n_d times the scales for the H and D quantities. That

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Mr. Nugent.

is, the actual space, say in inches, on the Q scale which corresponds to the logarithm of any number as 5, will be n_h times the actual corresponding space on the H scale. From an inspection of Equations (1) and (3) it is evident that, if the nomograph is to represent the typical Equation (1), n_h and n_d, respectively, must equal $\frac{x}{y}$ and $\frac{x+y}{y}$. Any convenient scale having been selected for the H line, the Q and D scales may now be determined.

The distances along any vertical, as the D line, from a point marked with a certain value of D to a value ten times as great,

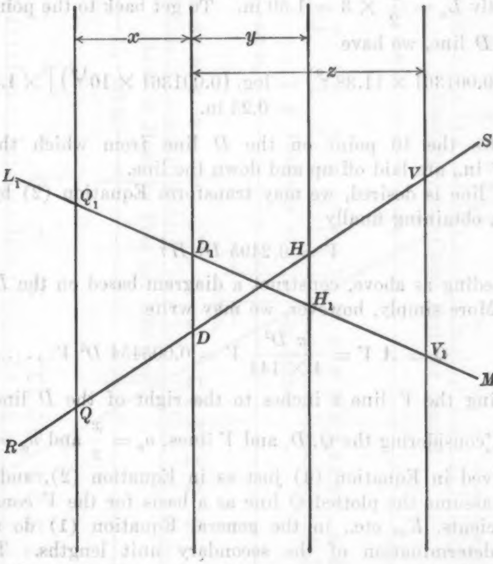


FIG. 24.

will be constant. These distances may be called the "secondary unit lengths" and denoted by L'_q, L'_d, L'_h. The "unit lengths", that is, the lengths corresponding to the logarithm of 10 (unity) are symbolized by L_q, L_d, and L_h.

The additional details of the construction may perhaps be best illustrated by the application of the theory to Equation (2) and its nomograph, Fig. 25. As shown in this figure, x and y are each assumed as 1.25 in. Consequently, $n_h = 1$ and $n_d = 2$.

In the formula, D occurs as $0.001361\ D^{\frac{19}{7}}$ and H as $H^{\frac{4}{7}}$. Q is found simply as Q. The scale assumed for the H line is 3 in. = unity.

Mr. Nugent. Therefore, $L'_h = \frac{4}{7} \times 3 = 1.71$ in. Similarly, $L'_q = 3.00$ in. and $L'_d = 4.07$ in. The spaces, L'_q and L'_h, are laid off up and down the Q and H lines from the zero line, RS.

Turning to the D line, we note that where it is intersected by RS, the value written for D, (D_0) must be such that

$$\log. \left(0.001361\, D_0^{\frac{19}{7}}\right) = 0$$

from which, $D_0 = 11.38$.

As shown above, the D scale is one-half the H scale, and consequently $L_d = \frac{1}{2} \times 3 = 1.50$ in. To get back to the point marked 10 on the D line, we have

$$\left[\log. \left(0.001361 \times 11.38^{\frac{19}{7}}\right) - \log. \left(0.001361 \times 10^{\frac{19}{7}}\right)\right] \times 1.50 \text{ in.}$$
$$= 0.23 \text{ in.}$$

This locates the 10 point on the D line from which the spaces, $L'_d = 4.07$ in., are laid off up and down the line.

If a V line is desired, we may transform Equation (2) by writing AV for Q, obtaining finally

$$V = 0.2495\, D^{\frac{5}{7}}\, H^{\frac{4}{7}}$$

and, proceeding as above, construct a diagram based on the D, H, and V lines. More simply, however, we may write

$$Q = A\,V = \frac{\pi\, D^2}{4 \times 144}\, V = 0.005454\, D^2\, V \ldots\ldots\ldots\ldots (4)$$

Assuming the V line z inches to the right of the D line, just as above, and considering the Q, D, and V lines, $n_v = \frac{x}{z}$ and $n_d = \frac{(x + z)}{z}$. Q is involved in Equation (4) just as in Equation (2), and we may, therefore, assume the plotted Q line as a basis for the V construction. The coefficients, K_d, etc., in the general Equation (1) do not enter into the determination of the secondary unit lengths. These are matters of the exponents, and, having determined L_d, $L'_d = b\, L_d$. Thus, considering the Q, D, and V lines,

$$L'_d = b\, L_d = b \frac{L_q}{\frac{(x + z)}{z}}, \text{ from which}$$

$$z = \frac{L'_d\, x}{b\, L_q - L'_d} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Using the values above given, from Equation (5), $z = 2.64$ in. Note that what has been done involves the assumption of a new L_d for the D line. Since the unit lengths have not been shown on the diagram, no confusion will be caused thereby.

Mr. Nugent.

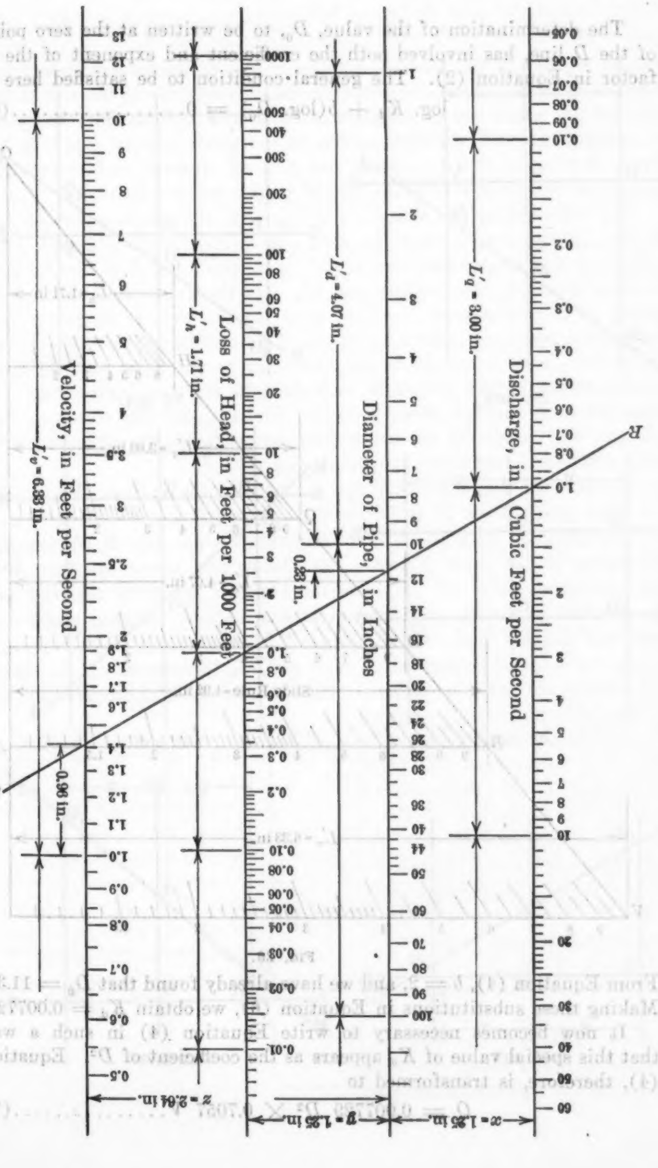


Fig. 25.

Mr. Nugent. The determination of the value, D_0, to be written at the zero point of the D line, has involved both the coefficient and exponent of the D factor in Equation (2). The general condition to be satisfied here is

$$\log. K_d + b(\log. D_0) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

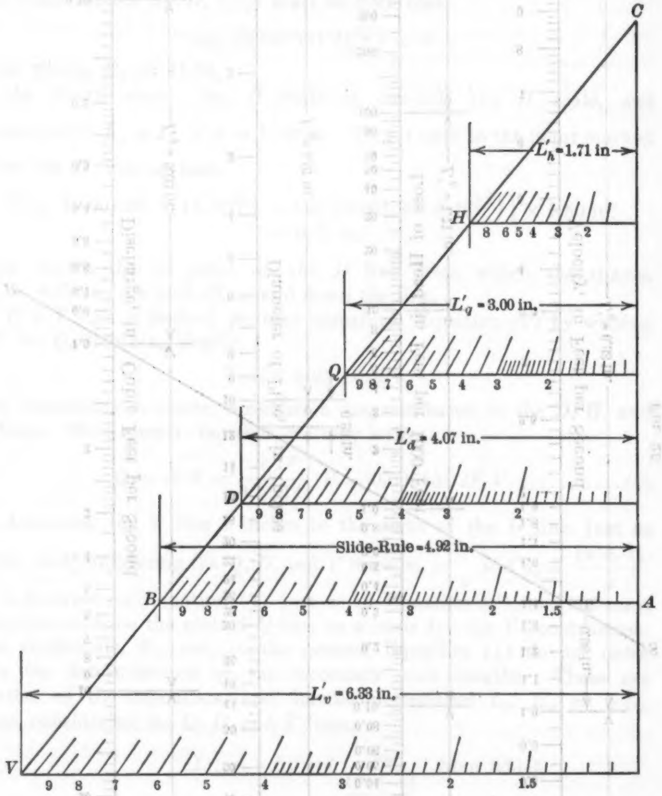


FIG. 26.

From Equation (4), $b = 2$, and we have already found that $D_0 = 11.38$. Making these substitutions in Equation (6), we obtain $K_d = 0.007729$.

It now becomes necessary to write Equation (4) in such a way that this special value of K_d appears as the coefficient of D^2. Equation (4), therefore, is transformed to

$$Q = 0.007729\ D^2 \times 0.7057\ V \quad \ldots\ldots\ldots\ldots\ldots (7)$$

Mr. Nugent.

We next have L_v $(= L'_v) = \frac{L_v}{n_v}$, from which, with the foregoing values, $L'_v = 4.28$ in. The value to be written at the intersection of the RS and E lines and the distance from this intersection to the end of the nearest L'_v space are determined as with the D line. H'_v is found to equal 1.417 and the distance up to the point marked 1 is 0.96 in. From this point the L'_v spaces are laid off upward and downward. Less than two such spaces are needed in the present diagram.

For subdividing the spaces, L'_v, L'_e, etc., the construction shown in Fig. 26 may conveniently be used. A right triangle is drawn with a base of any convenient length and an altitude equal to the longest of these spaces, in the present case L'_v. On the hypothenuse are chosen points at distances from the base equal to the various lengths to be subdivided and also to the unit length of an ordinary slide-rule. Lines are drawn from these points perpendicular to the base, and the line corresponding to the slide-rule is divided as on that instrument. A series of lines through these points of division and the vertex of the triangle will subdivide the other lines proportionally. The subdivisions are finally transferred to the nomograph.

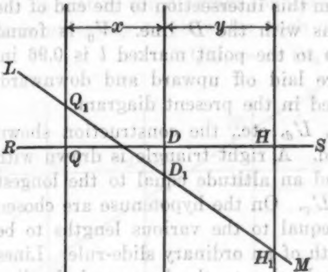


FIG. 27.

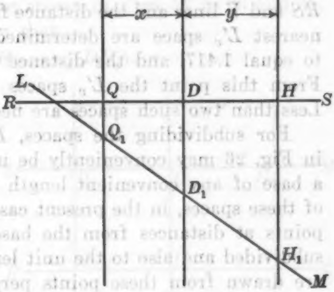


FIG. 28.

To consider the case where one or more of the various factors are less than unity, Figs. 27 to 35 show the nine possible positions of the straight-edge, or isopleth, LM. With each scale (Q, D, H, or E) increasing in the same direction both below and above the line, RS, the numerical value of the logarithm of any quantity read off on the diagram is always proportional to the distance between the point read and the RS line. In the general Equation (1), in solving for the Q factor, if its logarithm is plus and the logarithms of the other two factors are one plus and the other minus, the logarithm of the Q factor will be numerically equal to the difference between the other two. This corresponds to Figs. 27, 31, 32, and 35. The same thing is true if the logarithm of the Q factor is minus and the other two logarithms are one plus and the other minus. This corresponds to Figs. 28, 29, 30, and 33. If all the logarithms are negative, the logarithm of the Q factor is numerically equal to the sum of the other two; this corresponds to Fig. 34. This last case is entirely similar to that already discussed, where all the logarithms are posi-

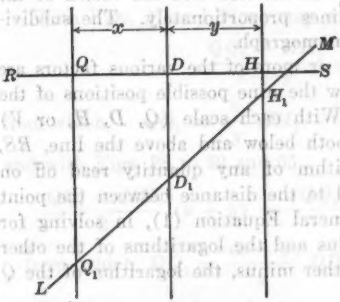


FIG. 29.

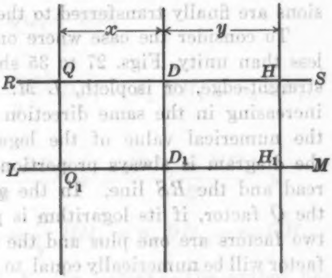


FIG. 30.

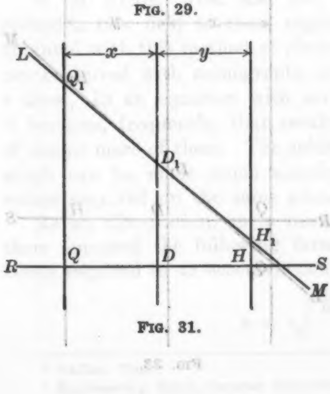


FIG. 31.

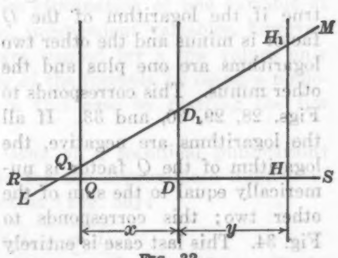


FIG. 32.

Mr. Nugent.

We next have $L_v (= L'_v) = \frac{L_q}{n_v}$, from which, with the foregoing values, $L'_v = 6.33$ in. The value to be written at the intersection of the RS and V lines and the distance from this intersection to the end of the nearest L'_v space are determined as with the D line. V_0 is found to equal 1.417, and the distance up to the point marked l is 0.96 in. From this point the L'_v spaces are laid off upward and downward. Less than two such spaces are needed in the present diagram.

For subdividing the spaces, L'_q, L'_d, etc., the construction shown in Fig. 26 may conveniently be used. A right triangle is drawn with a base of any convenient length and an altitude equal to the longest of these spaces, in the present case L'_v. On the hypotenuse are chosen points at distances from the base equal to the various lengths to be subdivided and also to the unit length of an ordinary slide-rule. Lines are drawn from these points perpendicular to the base, and the line corresponding to the slide-rule is divided as on that instrument. A series of lines through these points of division and the vertex of the triangle will subdivide the other lines proportionately. The subdivisions are finally transferred to the nomograph.

To consider the case where one or more of the various factors are less than unity, Figs. 27 to 35 show the nine possible positions of the straight-edge, or isopleth, $L\ M$. With each scale (Q, D, H, or V) increasing in the same direction both below and above the line, RS, the numerical value of the logarithm of any quantity read off on the diagram is always proportional to the distance between the point read and the RS line. In the general Equation (1), in solving for the Q factor, if its logarithm is plus and the logarithms of the other two factors are one plus and the other minus, the logarithm of the Q factor will be numerically equal to the difference between the other two. This corresponds to Figs. 27, 31, 32, and 35. The same thing is true if the logarithm of the Q factor is minus and the other two logarithms are one plus and the other minus. This corresponds to Figs. 28, 29, 30, and 33. If all the logarithms are negative, the logarithm of the Q factor is numerically equal to the sum of the other two; this corresponds to Fig. 34. This last case is entirely similar to that already discussed, where all the logarithms are posi-

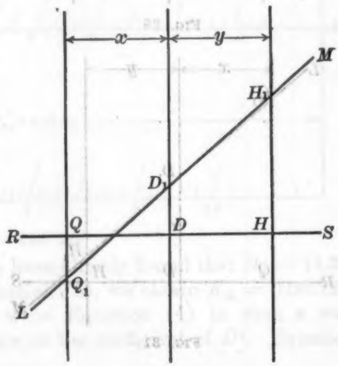


FIG. 33.

Mr. Nugent.

tive. Cases where one or more of the logarithmic factors become equal to zero, may be treated as special cases under some of those previously mentioned.

To justify the diagram for the cases shown in Figs. 27 to 33 and 35, it is necessary to show for each only that

$$QQ_1 = \pm\,(n_h\, HH_1 - n_d\, DD_1).$$

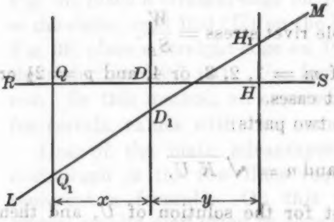


FIG. 34.

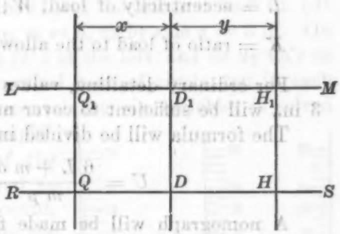


FIG. 35.

With Figs. 27 and 33 we may write

$$\frac{QQ_1 + HH_1}{x + y} = \frac{QQ_1 + DD_1}{x},$$

from which, $$QQ_1 = \frac{x}{y}\, HH_1 - \frac{(x + y)}{y}\, DD_1 = n_h\, HH_1 - n_d\, DD_1,$$

and with Figs. 28 to 32 and 35,

$$\frac{QQ_1 + HH_1}{2}\,(x + y) = \frac{(QQ_1 + DD_1)}{2}\,x + \frac{(DD_1 + HH_1)}{2}\,y,$$

from which, $$QQ_1 = \frac{(x + y)}{y}\, DD_1 - \frac{x}{y}\, HH_1 = n_d\, DD_1 - n_h\, HH_1.$$

Mr. Eliot.

W. M. ELIOT,* JUN. AM. SOC. C. E. (by letter).—The author has opened a new field to those engineers who hitherto have been unacquainted with this method of platting. As a general rule, any formula can be solved with nomographs by the elimination of one variable at a time. In an equation with several independent variables, however, it happens, frequently, that results are wanted for a few values only of one or more of them. The solution of the formula by using a nomograph can be made much simpler in this case by platting the few values required on the same graph.

As an illustration, in a recent issue of a technical publication† there appeared the following formula for the approximate number of rivets required in an eccentric connection (Fig. 36):

$$n = \sqrt{\frac{K\,(6\,L + m\,d)}{m\,p}}.$$

* Dallas, Tex.

† *Engineering News*, October 29th, 1914, p. 868.

Mr. Eliot.

n = number of horizontal rows of rivets;

m = number of vertical rows of rivets;

p = vertical rivet spacing;

d = horizontal rivet spacing;

L = eccentricity of load, W;

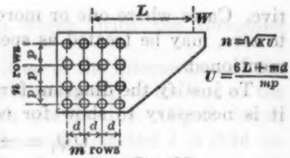


FIG. 36.

K = ratio of load to the allowable rivet stress $= \dfrac{W}{S_R}$.

For ordinary detailing, values of $m = 1, 2, 3$, or 4, and $p = 2\frac{1}{2}$ or 3 in., will be sufficient to cover most cases.

The formula will be divided into two parts:

$$U = \frac{6\,L + m\,d}{m\,p}, \text{ and } n = \sqrt{K\,U}.$$

A nomograph will be made first for the solution of U, and then one for n.

$$U = \frac{6\,L + 2\,d}{2 \times 2\frac{1}{2}}, \text{ when } m = 2, \text{ and } p = 2\tfrac{1}{2}$$

$$\frac{5}{8}\,U = \frac{3}{4}\,L + \frac{1}{4}\,d.$$

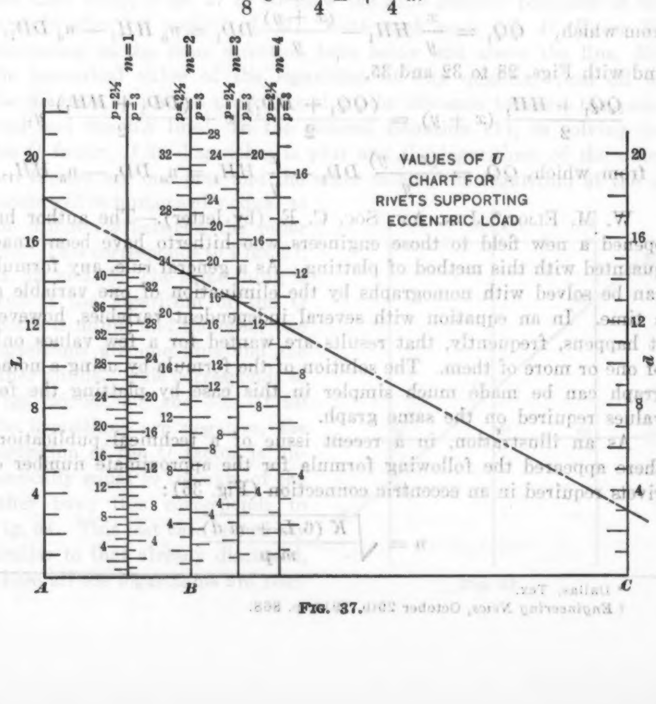


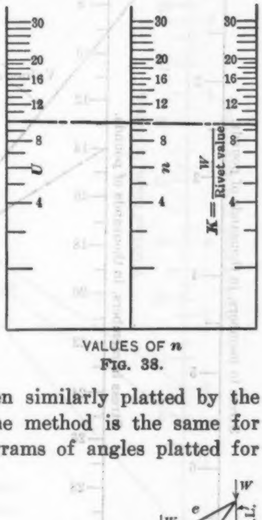
FIG. 37.

Mr. Eliot.

On the base line (Fig. 37), $AB = \frac{1}{4} AC$. The scale for $m = 2$, $p = 2\frac{1}{2}$, is five-eighths of that used for L and d. Other values of U are similarly platted.

Scales U, K, and n, are platted logarithmically (Fig. 38), lines for U and K being equidistant from n.

Given $K = 9\frac{1}{2}$, $L = 18$ in., $m = 4$, $p = 3$, and $d = 3$, to find n. On Fig. 37, place a straight-edge on 18 in. (L) at the left and on 3 in. (d) at the right; read 10.0 (U) on the line, $m = 4$, right side $p = 3$ in. On Fig. 38, place a straight-edge on 10.0 (U) at the left, and on $9\frac{1}{2}$ (K) at the right; read 9.7 (n). Use four vertical rows of rivets, ten in each row. By this method, an expression containing six variables is solved for certain values with two nomographs.



VALUES OF n
FIG. 38.

One of the main advantages of the nomograph is the few lines required to represent a formula. On this account, several results may often be shown on the same plat. The stresses in a roof truss (Fig. 39) of any slope and panel loading may be ascertained by the same general method as that used by the author for stresses in pins. As it is impracticable to use a separate graph for each member, they have been combined as shown in Fig. 40, two scales being used to give greater accuracy in the results. The stress in m 1 for a panel load of 2 000 lb., bevel of top chord 6 in 12, is 14 000 lb.

Other simple types of trusses have been similarly platted by the writer, but are not reproduced here, as the method is the same for all. By the use of these graphs, and diagrams of angles platted for strength in compression for different lengths, and for tension values with the proper size and number of holes deducted, it takes only a few minutes to ascertain the make-up of a roof truss for an ordinary mill building. An excess vertical load is assumed to take care of wind stresses, but this need not be the same for all members.

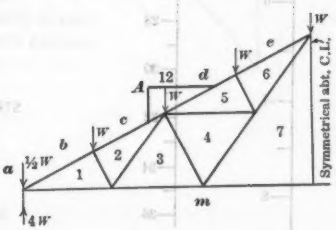


FIG. 39.

The writer does not believe the author's chart for reinforced concrete beams will be used to any extent by engineers, as the diagrams*

* *Transactions*, Am. Soc. C. E., Vol. LVI, p. 360.

Mr. Eliot.

by Arthur W. French, M. Am. Soc. C. E., cover the same ground in a simple manner, and are now in general use. The two nomographs which follow, though only for stresses of 650 and 16 000 lb. per sq. in.

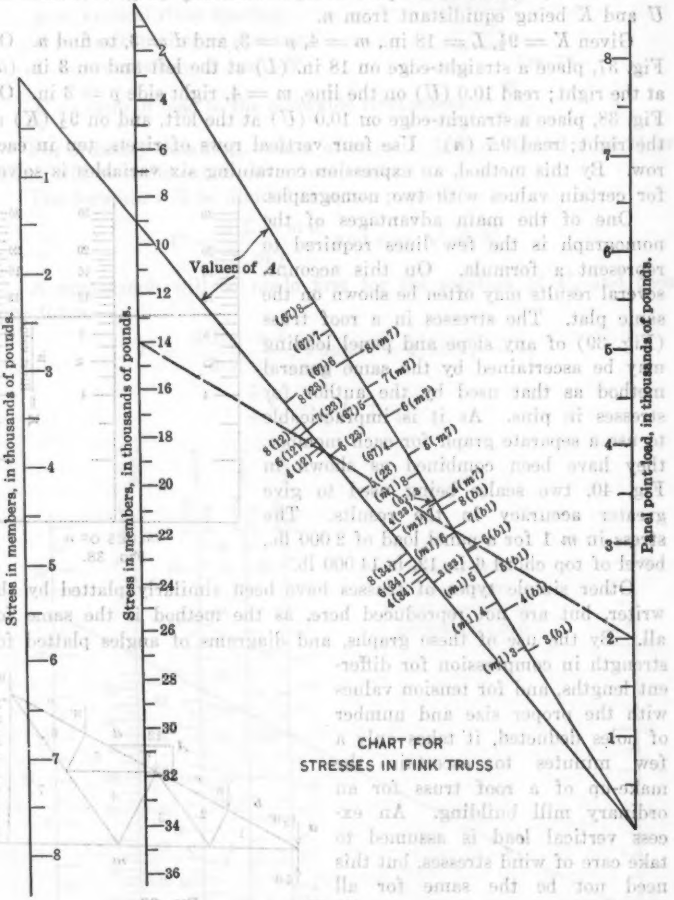


FIG. 40.

on the concrete and steel, respectively, give more direct results than the one proposed by the author. The diagram, Fig. 41, gives the size of beam and area of steel required to resist a given moment, when

the concrete and steel are each stressed to the working limit. For the case shown, a moment of 428 000 in-lb. requires a beam 10 by 20 in. (effective depth), and an area of steel = 1.54 sq. in. Mr. Elliot.

Often the size of beam is fixed, and it is desired to know the quantity of steel required for a given resisting moment. This is

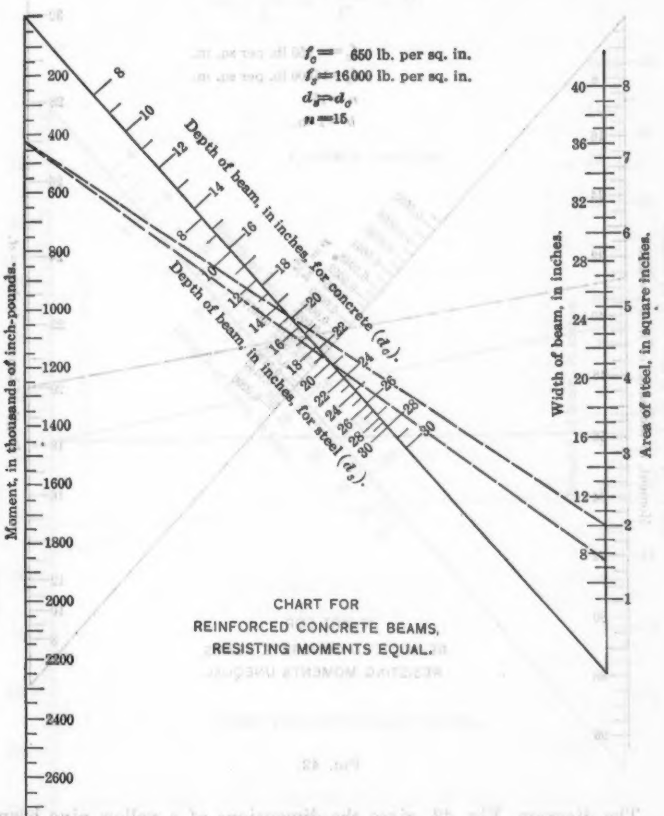


FIG. 41.

given by the nomograph, Fig. 42; the resisting moment is determined by the steel being stressed to the working limit for values of p less than 0.0077, and by the concrete for values of p greater than 0.0077. Given a beam, 10 by 20 in. (effective depth) to resist a moment of

Mr. Eliot. 350 000 in-lb.: Place a straight-edge on the moment 35 000 lb. (for 1 in. in width) and on $d = 20$; read $p = 0.0062$. Area of steel required $= p\ b\ d = 1.24$ sq. in. One designing many beams can well afford to make similar nomographs for the stresses he uses, as little time is required.

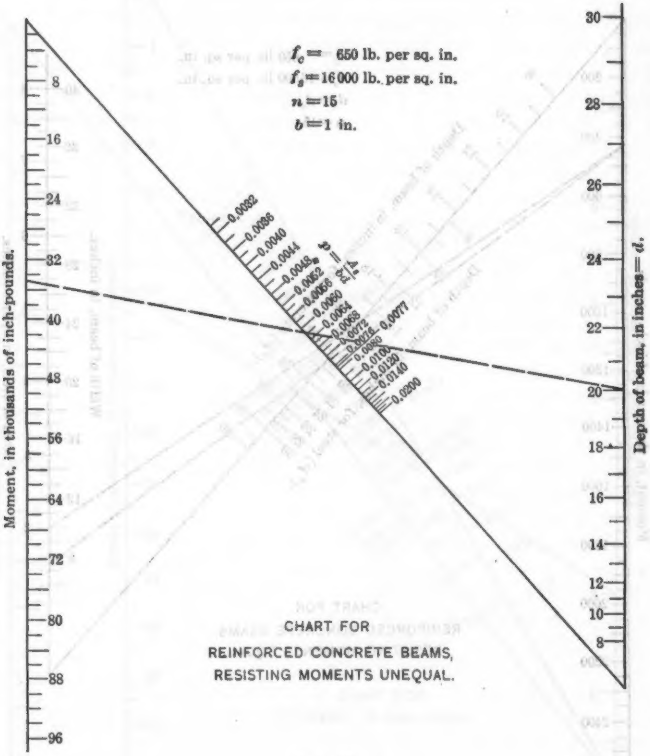


FIG. 42.

The diagram, Fig. 43, gives the dimensions of a yellow pine beam directly, when the moment is known. If the total uniform load and span are given, two isopleths are required to ascertain the size of beam. In the example shown, a load of 19 000 lb. on a span of 20 ft. requires a beam 10 by 16 in. (actual size $9\frac{1}{2}$ by $15\frac{1}{2}$ in.). The moment itself need not be read.

The pitch of rivets connecting the web and flange of a plate girder (see Fig. 44) is given approximately by the formula: Mr. Eliot.

$$p = \frac{R\,h}{V},$$

$$\text{or, } \frac{10\,p}{h} = \frac{10\,R}{V} = Q.$$

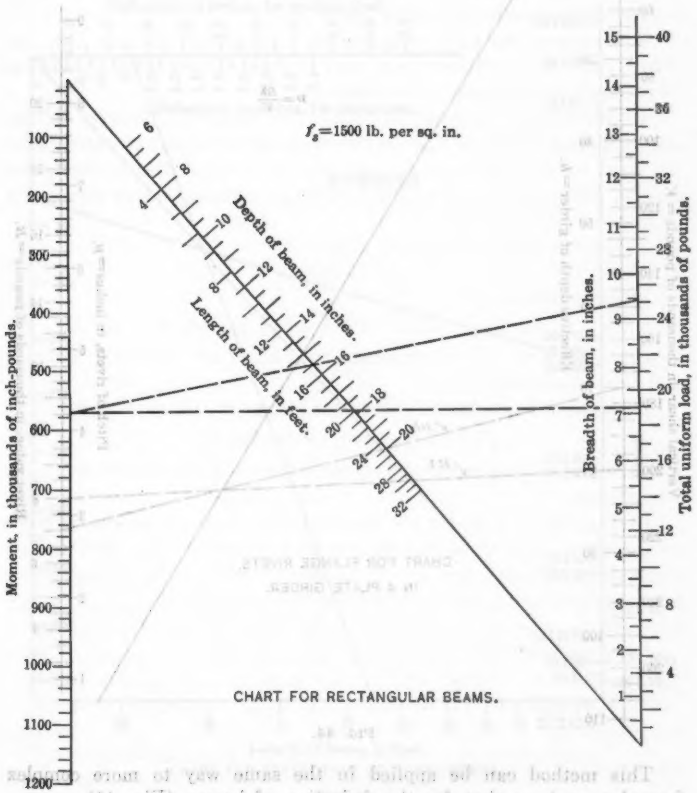


FIG. 43.

The isopleths joining p and h and R and V, intersect on the diagonal line for Q connecting the ends of the scales. Thus the pitch is 2.8 in. for a vertical shear of 200 000 lb. on a girder having an effective depth of 70 in., with a rivet value of 8 000 lb. As this formula

Mr. Eliot. gives a pitch that is a little low, correction can be made to some extent by using the total for the effective depth of the girder. As the values of Q are not platted, nomographs for equations of this type are easily made.

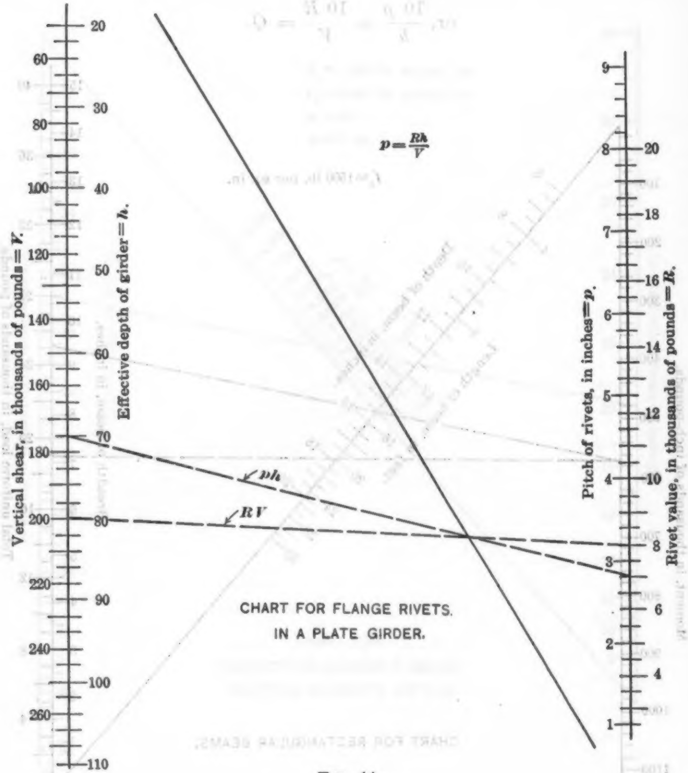


FIG. 44.

This method can be applied in the same way to more complex formulas, such as that for the deflection of beams (Fig. 45).

$$\Delta = \frac{5\ W\ L^3}{384\ E\ I},$$

$$\text{or,}\ \frac{384 \times 58\,000\ \Delta}{L^3} = \frac{W}{100\ I} = Q,$$

Mr. Eliot.

when $E = 29\,000\,000$ for steel;
$\Delta =$ deflection;
$L =$ span;
$W =$ total uniform load;
$I =$ moment of inertia.

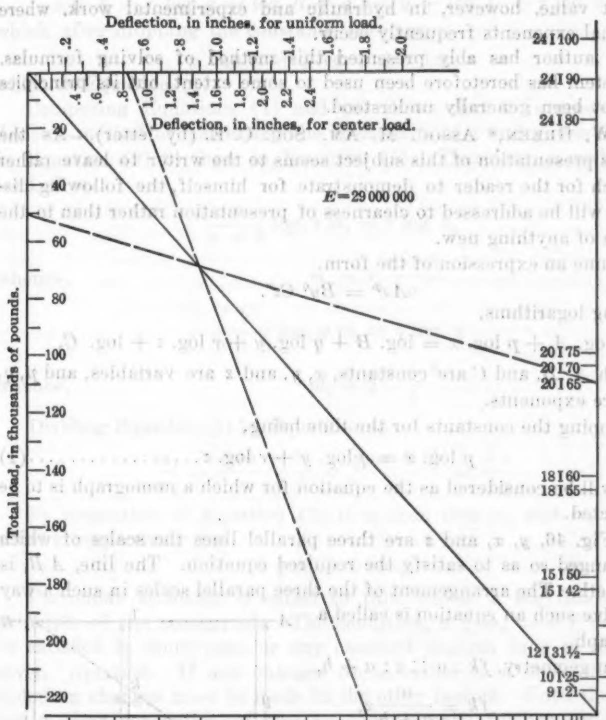
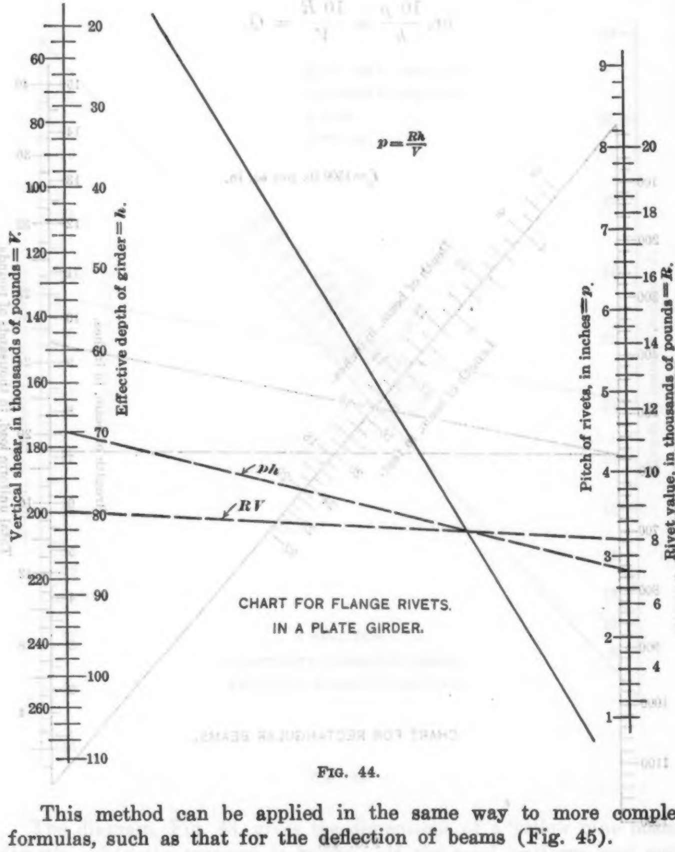


CHART FOR DEFLECTION OF STEEL BEAMS.

FIG. 45.

The cubes of L are used, and the beams are written at their respective moments of inertia when the members are platted. The deflections for a concentrated center load are similarly shown. A 20-in. I-beam of 65 lb. per ft., with a total uniform load of 50 000 lb. on a span of 25 ft., has a deflection of 0.52 in.

Mr. Eliot. gives a pitch that is a little low, correction can be made to some extent by using the total for the effective depth of the girder. As the values of Q are not platted, nomographs for equations of this type are easily made.



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$$\Delta = \frac{5\ W\ L^3}{384\ E\ I},$$

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Mr. Eliot.

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$\Delta =$ deflection;
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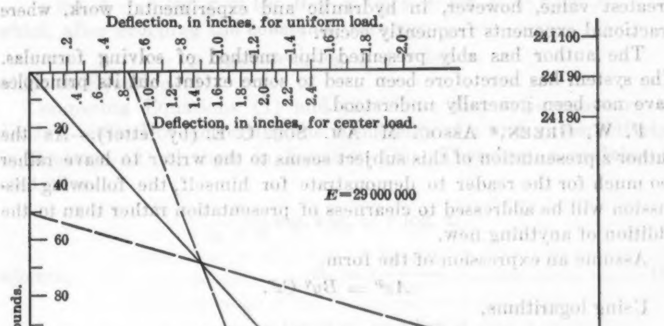
$W =$ total uniform load;
$I =$ moment of inertia.



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Mr. Eliot. The writer, in presenting these nomographic solutions for formulas used in structural engineering, has not given the details of construction in each case, as the method will be apparent to any one who has read the paper. The alignment diagram is very useful in this field, even though many of the formulas are readily solved by direct computation. It would seem that this method would be of greatest value, however, in hydraulic and experimental work, where fractional exponents frequently occur.

The author has ably presented this method of solving formulas. The system has heretofore been used to some extent, but its principles have not been generally understood.

Mr. Green. F. W. GREEN,* ASSOC. M. AM. SOC. C. E. (by letter).—As the author's presentation of this subject seems to the writer to leave rather too much for the reader to demonstrate for himself, the following discussion will be addressed to clearness of presentation rather than to the addition of anything new.

Assume an expression of the form,

$$A x^p = B y^q C z^r.$$

Using logarithms,

$$\log. A + p \log. x = \log. B + q \log. y + r \log. z + \log. C,$$

in which A, B, and C are constants, x, y, and z are variables, and p, q, and r are exponents.

Dropping the constants for the time being,

$$p \log. x = q \log. y + r \log. z \quad \ldots\ldots\ldots\ldots (1)$$

which will be considered as the equation for which a nomograph is to be constructed.

In Fig. 46, y, x, and z are three parallel lines the scales of which are arranged so as to satisfy the required equation. The line, $A\,B$, is an isopleth. The arrangement of the three parallel scales in such a way as to solve such an equation is called a nomograph.

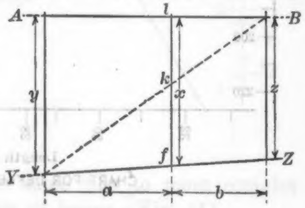

FIG. 46.

From geometry, $fk : a :: z : a + b$,

whence $$fk = \frac{a}{a+b} z,$$

and, similarly, $kl : y :: b : a + b$,

whence, $$kl = \frac{b}{a+b} y.$$

As $fk + kl = x$,

therefore, $$x = \frac{a}{a+b} z + \frac{b}{a+b} y \quad \ldots\ldots\ldots\ldots (2)$$

Equation (2) shows the relation of the scales of values for the variables, which is required in order to satisfy the given equation.

* Stamps, Ark.

The expression assumed was, Mr. Green.

$$A x^p = B y^q C z^r,$$

whereas Equation (2), in the right-hand member, involves addition instead of multiplication. Therefore, logarithms must be used in order to make Equation (2) applicable, thus,

$$\log. A + p \log. x = \log. B + q \log. y + \log. C + r \log. z,$$

which, after dropping the constants, becomes,

$$p \log. x = q \log. y + r \log. z$$

which is Equation (1).

Comparing Equations (1) and (2) shows that it is necessary to devise a method of converting the coefficients of z and y, respectively, in Equation (2) to those in Equation (1). Therefore, a modulus is used, thus:

$$\frac{a}{a+b} \log. z \, m_1 = r \log. z,$$

whence,
$$m_1 = r \frac{a+b}{a} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\frac{b}{a+b} \log. y \, m_2 = q \log. y$$

whence,
$$m_2 = q \frac{a+b}{b} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Dividing Equation (3) by Equation (4),

$$\frac{m_1}{m_2} = \frac{r\,(a+b)}{a} \div \frac{q\,(a+b)}{b} = \frac{b\,r}{a\,q} \ldots\ldots\ldots\ldots (5)$$

By inspection of Equation (5) it is seen that m_1 and m_2 vary as a and b, respectively; or

$$a\,q\,m_1 = b\,r\,m_2.$$

Particular attention is called to this, which is the fundamental principle of the nomograph. The condition, $a\,q\,m_1 = b\,r\,m_2$, must be satisfied in every case, or any assumed isopleth fails to solve the given equation. If any changes occur, either in a or b, the corresponding changes must be made in the other factors. Until the reader has a clear conception of this he cannot make further satisfactory progress.

To illustrate this principle, one example will be worked out in detail. The author refers to a modification of the Williams-Hazen formula as

$$v = 0.0796\, d^{0.63}\, h^{0.54}.$$

For convenience in applying the formula, this will be written,

$$x = 0.0796\, y^{0.63}\, z^{0.54} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Using logarithms,

$$\log. x = \log. 0.0796 + 0.63 \log. y + 0.54 \log. z$$

$$\log. x - \log. 0.0796 = 0.63 \log. y + 0.54 \log. z.$$

Mr. Green. a and b may be assumed as having any arbitrary value whatever. In this case, the exponents will be used for such arbitrary values, and it will be assumed that $a = 63$ and $b = 54$.

Then, from Equation (2),

$$\log. x - \log. 0.0796 = \frac{63}{117} \log. z + \frac{54}{117} \log. y.$$

Substituting in Equation (5) and solving for $\frac{m_1}{m_2}$,

$$\frac{m_1}{m_2} = \frac{b\ r}{a\ q} = \frac{0.54 \times 54}{0.63 \times 63} = \frac{29.16}{39.69} = \frac{1}{1.361}.$$

Draw three parallel lines for the x, y, and z scales, respectively, with the distance from y to $x = \frac{63}{117}$ of that from y to z. If the highest values desired are for $y = 60$, and $z = 100$, an isopleth may be drawn across these scales near the top (as the line, $A\,B$, in Fig. 46), and the intersection with y marked 60 and that with z marked 100. Substituting these values in Equation (6) and solving for x, gives $x = 12.62$, which value is marked on the x scale, similarly.

Next make a table (Table 1) in order to obtain values for plotting the various scales.

TABLE 1.

y (1)	log. y (2)	m_2 log. y = 1.361 log. y (3)	z (4)	log. z (5)	m_1 log. z = 1 log. z (6)	x (7)	log. x (8)
60	1.778	2.420	100	2.000	2.000	12.62	1.101
50	1.699	2.312	90	1.954	1.954	12.00	1.079
40	1.602	2.180	80	1.903	1.903	11.00	1.041
30	1.477	2.008	70	1.845	1.845	10.00	1.000
20	1.301	1.770	60	1.778	1.778	9.00	0.954
10	1.000	1.361	50	1.699	1.699	etc.	etc.
9	0.954	1.298	40	1.602	1.602	....	....
8	0.903	1.228	30	1.477	1.477	....	....
7	0.845	1.150	20	1.301	1.301	....	....
6	0.778	1.058	10	1.000	1.000	....	....
5	0.699	0.952	9	0.954	0.954	....	....
4	0.602	0.820	etc.	etc.	etc.	....	....
3	0.477	0.649	....	....	....	....	....
2	0.301	0.410	....	....	....	....	....
1	0.000	0.000	1	0.000	0.000	1.00	0.000

Using any convenient scale (as the 20th) plot the values of y from the items in Column (3), and, similarly, with the same scale, plot the values of z from the items in Column (6).

Using the same scale, the values of $\log. x - \log. 0.0796$ are plotted as follows:

$$\log.\ 12.62 = 1.101$$

$$\log.\ 0.0796 = \bar{2}.901 = -2 + 0.901 = -1.099$$

$$\log.\ 12.62 - \log.\ 0.0796 = 1.101 - (-1.099) = 2.200$$

Mr. Green.

The point on the x scale where log. x — log. 0.0796 = 1 is, therefore, 2.200 units below the isopleth intersecting the x scale at a value for $x = 12.62$; but, as it is necessary to plot values of x, and not values of $\frac{x}{0.0796}$, the point must be found which indicates $x = 1$. Accordingly, the value of log. 0.0796 is subtracted by adding 1.099 to the point (2.200 units below the value of $x = 12.62$) where log. x — log. 0.0796 = 1. Marking this latter point 1, the values are scaled from the items in Column (8).

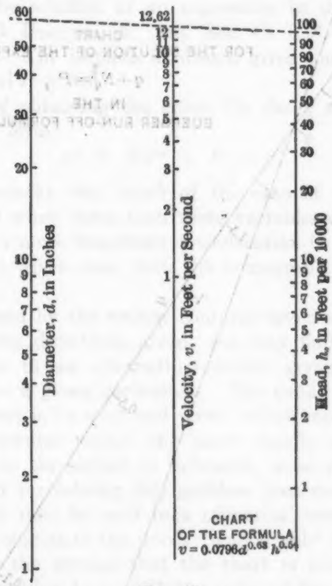

FIG. 47.

In any case, the numerical coefficients of any of the variables may be treated in a similar way. The method of working with exponents has already been developed quite fully. Fig. 47 is the chart made from the foregoing description.

Mr. Strachan.

R. C. STRACHAN,* M. AM. SOC. C. E. (by letter).—The discussion has brought out a number of interesting points, among which may be mentioned the demonstration by Mr. Nugent of the fact, suggested but not enlarged upon in the paper, that some formulas may be charted

* Richmond Hill, N. Y.

Mr. Strachan.

in more than one way. This is a distinct advantage, as it enables one to choose the form best adapted to his purposes.

Mr. Eliot has well shown how the use of natural scales may be extended to several formulas in addition to those similarly charted

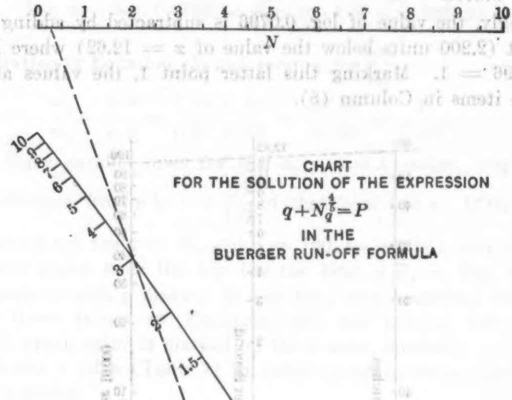

FIG. 48.

in the paper. In reference to this the writer would emphasize the fact that the nomograph is a generalization which includes the many forms of the slide-rule as particular cases. The slide-rule is essentially an instrument designed for adding logarithms graphically; and its

parallel scales are in such relative positions that all isopleths—placed with the runner—are perpendicular to them. Mr. Strachan.

The broader principle, however, enables one to use either logarithmic or natural scales, and to perform all arithmetical operations graphically. Scales may be laid down either on straight lines or curves, and they need not be parallel.

The result of any one of the many problems which are daily solved with the aid of the slide-rule, as well as the results of others not adapted to this means of solution, may be read with sufficient precision from a properly constructed chart.

Fig. 48, for the solution of an expression in the run-off formula proposed by C. B. Buerger, M. Am. Soc. C. E.,* illustrates the use of a curved scale. The isopleth as drawn gives the value of $q = 3.0$ when $N = 0.17$ and $P = 3.4$.

The method of obtaining the curve for the q scale applies to all expressions of the form:

$$x^n + Nx^m + P = 0.$$

Mr. Hazen remarks that much of the ease of application of the nomograph is lost when more than three variables enter the equation. This is true; but a more important consideration is the ease of obtaining solutions, in a given case, with the nomograph, as compared with other methods.

Fig. 49, designed by the writer, contains seven variables, and with it, by following the directions given, one may find the required size of wire conductor in an alternating-current system when the "line drop" is limited to a given percentage. The usual and accepted solution of this problem is by trial and error; which means that the better the guess the computer makes, the more rapidly will he obtain the answer. So far as the writer is informed, none of the many other diagrams invented for solving this problem does more than furnish a factor which must then be used in a numerical computation.

Mr. Eckersley objects to the words "nomograph" and "nomography" as misnomers, on the ground that the chart is not a "written law". Though this has nothing to do with the value of the charts, and though the writer is not in any way responsible for the name of the science, it is his opinion that the word "nomography" is the most appropriate one that could have been selected.

A mathematical law may be written in more than one way, as a statute law may be written in more than one language. With one object in view, the law is written as the graph or locus, and these words are perfectly well understood to refer to a certain method of representation. No exposition of the elementary principles of analytical geometry is required to make this clear.

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 1139.

Mr. Strachan.

When, with a distinctly different object in view, a new method of writing the law of the equation was invented, it was eminently proper that a distinctive yet descriptive word should be used in referring to it.

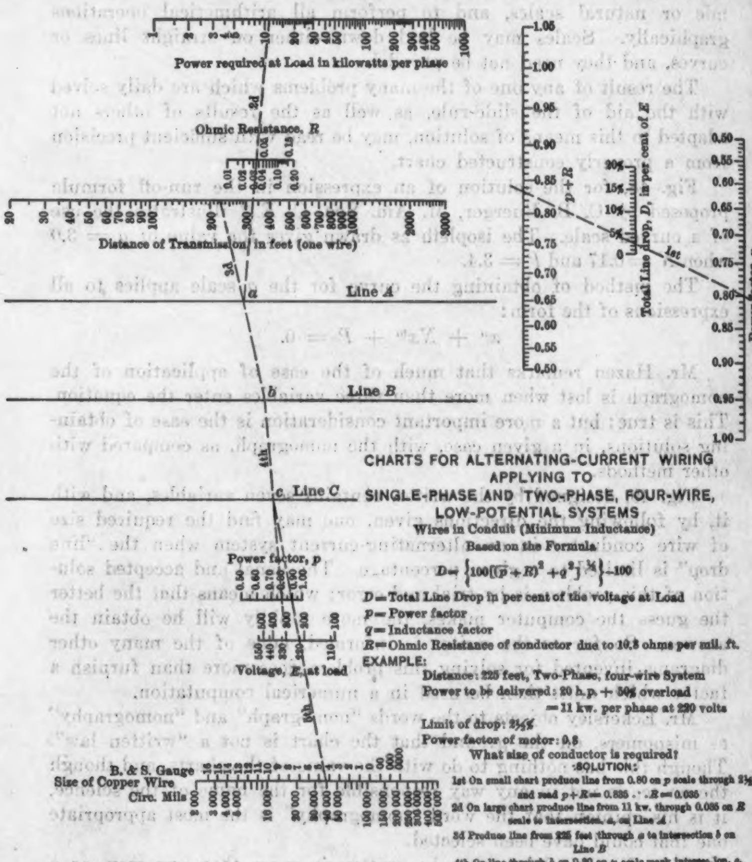

FIG. 49.

The terms "graphical arithmetic", "graphic chart", and "metrical table" seem crude when compared with the word selected by the able originator of the method.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1334

THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA*

BY E. L. SAYERS AND A. C. POLK, MEMBERS, AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. JAMES H. HARLOW, R. D. COOMBS, HUGH L. COOPER, HEINRICH HOMBERGER, H. BIRCHARD TAYLOR, SPENCER MILLER, W. E. MITCHELL, GARDNER S. WILLIAMS, AND E. L. SAYERS AND A. C. POLK.

SYNOPSIS.

The object of this paper is to describe the design and construction of a low-head hydro-electric plant recently completed in Alabama, and thereby bring out a discussion, if possible, on the latest ideas in the practice of developing water-power under low heads, the latest features in hydraulic turbine design, modern methods of transmission, and the construction of such plants.

The Alabama Traction, Light, and Power Company was organized in 1912 for the purpose of developing the water-powers of Alabama. The Company acquired several public service companies operating steam generating plants, street railways, gas plants, etc., and several companies owning undeveloped water-power sites. Among the latter was the Alabama Power Company, which owns a site known as "Lock 12" on the Coosa River at a central point in the State.

This plant will form the hub of an extensive system of power

* Presented at the meeting of November 4th, 1914.

plants capable of developing 24-hour, 80% efficiency power in excess of 500 000 h.p.

The foundations of the power-house are built transversely to the axis of the river and form an integral part of the dam. The length of spillway, power-house, and abutments, over all, is 1 530 ft. 5½ in. The power-house substructure is 303 ft. 4 in. long, 136 ft. 5 in. wide, and contains provision for six units. Four units constitute the present installation, and the superstructure is built with a temporary wall at the river end so that it can be extended to cover all six.

The spillway forming the principal part of the dam has a net crest length of 780 ft., divided into twenty-six sections, each 30 ft. long, separated by concrete piers, 6 ft. long on the axis of the dam. On the crest of each section of spillway there is placed between piers a gate, 30 ft. long and 14 ft. high, of a modified Stoney type. The height of the dam from the average elevation of the original river bed to the elevation of the flow line of the reservoir is 77 ft.

In all, there are approximately 190 000 cu. yd. of concrete in the power-house foundations and the dam. The first concrete was placed on April 12th, 1913; it was all placed by February 26th, 1914; and commercial power was turned out for the first time on April 12th, 1914, just one year, to the day, from the date of placing the first concrete.

The installation is made up of four vertical, single-runner turbines, guaranteed to deliver to the shaft, under 68 ft. head, at 100 rev. per min., 17 500 h.p. One feature of these units, as described in the paper, is that they are entirely self-contained, that is, they can be built up on an assembly floor from the foundation ring to the top of the direct-connected exciter above the generator, without requiring any lateral support.

Another feature of the development, as described in the paper, is the use of an improved volute casing, through which the water passes from the penstocks to the wheels. These scroll casings were complicated in form, and were moulded directly in the concrete of the power-house foundations, without the use of a steel lining.

After giving a brief description of the general system of the Alabama Traction, Light, and Power Company and of the power market to be served, the paper describes the Coosa River and the cli-

matic conditions under which the work was done, so that a better idea can be had by those not familiar with the locality.

The machinery is fully described, greater detail being given in the case of the turbines than for the electrical equipment, discussions of which are constantly appearing in papers of electrical societies and in the electrical technical press.

In connection with the construction of the plant, the materials used and their sources, the general scheme of handling the work and its prosecution, are dealt with in detail. The selection of plant, its layout, and the construction of camps are described, and improvements which experience showed could have been used, are noted.

The concrete forms used in the power-house foundations were necessarily complicated, and the details of their design and construction are shown. The various methods used in placing concrete and the success attending their use are described.

At the end of the paper, the design and construction of transmission lines and sub-stations are dealt with briefly.

INTRODUCTORY.

In the last 10 years the progress made in methods of generating electric power and transmitting it over long distances to markets has been very marked, and although more or less complete descriptions of hydro-electric developments have been appearing in various technical papers, there has been no extended discussion of modern methods before this Society. This fact appears the more strange because at the present time a large amount of work is being done in developing the water-powers of the United States, and this branch of the Profession is fast becoming one of the most important.

No extraordinary features are claimed for the work described. The plant develops power under the moderate head of 68 ft., the installation of 70 000 h.p. is not remarkable, and no great difficulties in construction were met. It is believed, however, that it represents the latest improvements in equipment, and that a full discussion of this development and others of similar proportions will be of practical value to the Profession by bringing before it the latest practice in developing water-power under low heads, the latest features in hydraulic turbine design, modern methods of transmission

of high-potential current, and methods of construction which have been tried and found successful.

General System of the Alabama Traction, Light, and Power Company.

The Alabama Traction, Light, and Power Company, organized in 1912, acquired the properties, rights, and franchises of several public service companies in Alabama, and of several companies owning water-power sites in the State. Among the latter were the Alabama Power Company, with several undeveloped sites on the Coosa River; the Alabama Interstate Power Company, with a site on the Tallapoosa River; the Little River Power Company, with a site on Little River; and the Muscle Shoals Hydro-Electric Power Company, owning three sites on the Tennessee River. These sites represent a possible development in excess of 500 000 h.p. continuous 24-hour power.

The developments proposed by the Company on the Coosa are at the sites selected by the United States Government engineers for dams and locks for the improvement of the river. Fig. 1 is a profile of the Coosa River covering that portion affected by the proposed developments. The dam farthest south, at which power could be developed, is at a point 7½ miles above Wetumpka. This site, known as Lock 18, has an effective head of 63 ft., and the dam would raise the water to Elevation 254 (U. S. Army datum) from Elevation 190. At Lock 15, the next site, 19½ miles above Wetumpka, the water would be raised from Elevation 254 to Elevation 294, giving an effective head of about 39 ft. At a point about 34 miles above Wetumpka, the dam at Lock 14 would raise the water to Elevation 352, giving an effective head of about 57 ft. Lock 12 is 40 miles above Wetumpka, and here the water level is raised to Elevation 420, making the effective head of 67 ft. These sites are all shown on the general map, Fig. 2.

The proposed development on the Tallapoosa River is at Cherokee Bluffs, about 30 miles northeast of Montgomery. Here a dam 180 ft. high would back the water up 30 miles in the river, making one of the largest storage reservoirs in the country, the area being 34 000 acres.

The Little River development is in the northeastern part of the State (see Fig. 2), about 33 miles northeast of Gadsden. The devel-

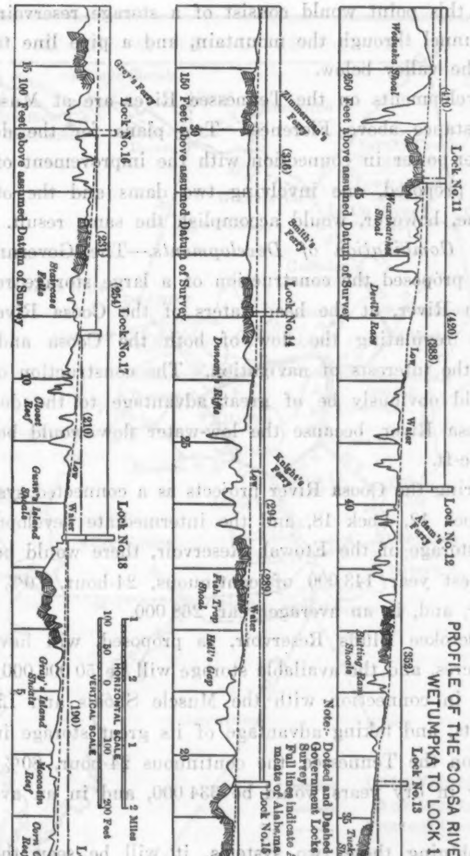
PROFILE OF THE COOSA RIVER FROM
WETUMPKA TO LOCK 11
Note: Dotted and Dashed lines indicate Proposed Government Locks from the Cavanaugh Survey
Full lines indicate Alternative Developments of Alabama Power Co.
Lock No.11
Lock No.12
Lock No.13
New Lock 14
Lock No.14
Lock No.15
Lock No.16
Lock No.17
Lock No.18
Lock No.19
Lock No.20 Formerly 31
250 Feet above assumed Datum of Survey
150 Feet above assumed Datum of Survey
100 Feet above assumed Datum of Survey
Low Water
HORIZONTAL SCALE
VERTICAL SCALE
Wetumpka Wagon Bridge

FIG. 1.

opment at this point would consist of a storage reservoir on Little River, a tunnel through the mountain, and a pipe line to a power-house in the valley below.

The developments on the Tennessee River are at Muscle Shoals, a short distance above Florence. Two plans for the development of the water-power in connection with the improvement of the river have been proposed, one involving two dams and the other three, each scheme, however, would accomplish the same result.

Possible Combination of Developments.—The Government engineers have proposed the construction of a large storage reservoir on the Etowah River, at the head-waters of the Coosa River, for the purpose of regulating the flow of both the Coosa and Alabama Rivers, in the interests of navigation. The construction of this reservoir would obviously be of great advantage to the developments on the Coosa River, because the low-water flow would be increased to 4 700 sec-ft.

Considering the Coosa River projects as a connected system, tying together Lock 12, Lock 18, and the intermediate developments, and using the storage of the Etowah Reservoir, there would be produced in the dryest year 143 000 of continuous, 24-hour, 80% efficiency, horse-power, and, in an average year, 268 000.

The Cherokee Bluffs Reservoir, as proposed, will have an area of 34 000 acres, and the available storage will be 50 000 000 000 cu. ft. Using this in connection with the Muscle Shoals and Little River developments, and taking advantage of its great storage in times of low water on the Tennessee, the continuous 24-hour, 80% efficiency, horse-power in dry years would be 334 000, and in an average year 505 000.

By combining these two systems, it will be seen that, in the ultimate development, there is obtainable 24-hour, 80% efficiency, to an amount in excess of 550 000 h.p., with a 100% load factor. On the ordinary commercial load factor there would be required a capacity of about 1 000 000 h.p., which is probably in excess of the amount that the State will use for many years to come.

A knowledge of these facts is only desirable in that they partly explain the reasons for the adoption of certain types of construction as being part of an extensive proposed system.

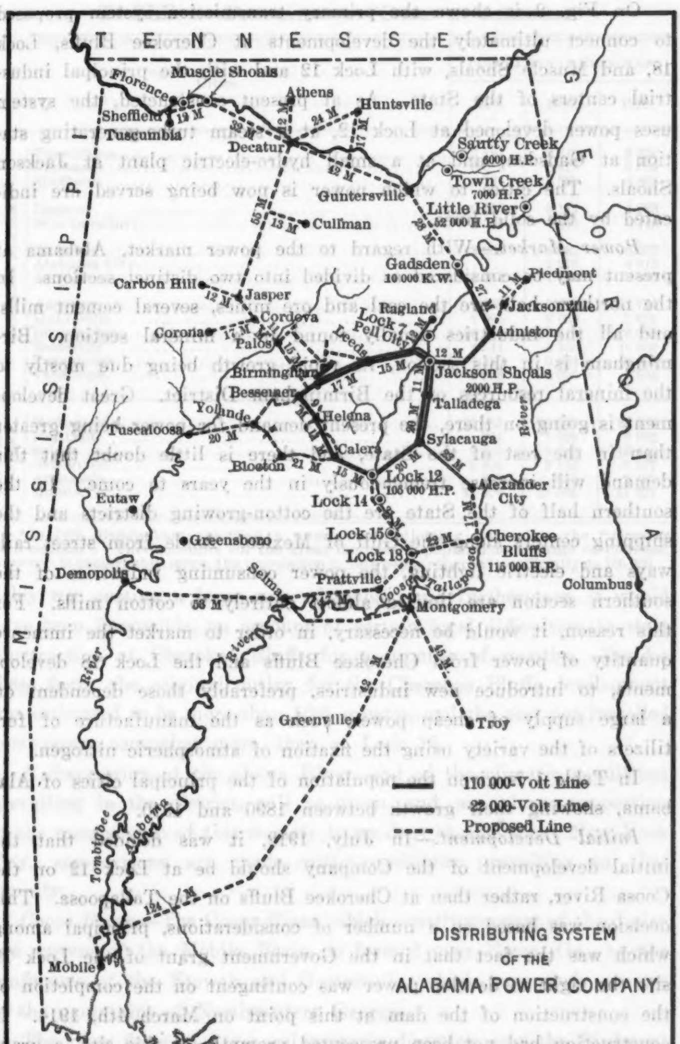
T E N N E S S E E
G E O R G I A
M I S S I S S I P P I
Muscle Shoals
Florence
Sheffield
Tuscumbia
Athens
Huntsville
Decatur
Guntersville
Sauty Creek
6000 H.P.
Town Creek
7000 H.P.
Little River
52 000 H.P.
Cullman
Gadsden
10 000 K.W.
Piedmont
Carbon Hill
Jasper
Jacksonville
Ragland
Lock 7
Pell City
Anniston
Cordova
Corona
Palos
Leeds
Birmingham
Jackson Shoals
2000 H.P.
Bessemer
Talladega
Yolande
Helena
Tuscaloosa
Sylacauga
Calera
Blocton
Lock 12
105 000 H.P.
Alexander City
Eutaw
Lock 14
Lock 15
Cherokee Bluffs
115 000 H.P.
Greensboro
Lock 18
Demopolis
Prattville
Columbus
Selma
Coosa
Tallapoosa
Montgomery
River
Greenville
Troy
Tombigbee
Alabama
Mobile
110 000-Volt Line
22 000-Volt Line
Proposed Line
DISTRIBUTING SYSTEM
OF THE
ALABAMA POWER COMPANY

FIG. 2.

On Fig. 2 is shown the primary transmission system proposed to connect ultimately the developments at Cherokee Bluffs, Lock 18, and Muscle Shoals, with Lock 12 and with the principal industrial centers of the State. As at present constructed, the system uses power developed at Lock 12, at a steam turbo-generating station at Gadsden, and at a small hydro-electric plant at Jackson Shoals. The cities to which power is now being served are indicated by the solid lines.

Power Market.—With regard to the power market, Alabama at present may be considered as divided into two distinct sections. In the northern half are the coal and ore mines, several cement mills, and all the industries usually found in a mineral section. Birmingham is in this section, its rapid growth being due mostly to the mineral resources of the Birmingham District. Great development is going on there, the present demand for power being greater than in the rest of the State, and there is little doubt that this demand will increase tremendously in the years to come. In the southern half of the State are the cotton-growing districts and the shipping centers along the Gulf of Mexico. Aside from street railways and electric lighting, the power consuming industries of the southern section are limited almost entirely to cotton mills. For this reason, it would be necessary, in order to market the immense quantity of power from Cherokee Bluffs and the Lock 18 developments, to introduce new industries, preferably those dependent on a large supply of cheap power, such as the manufacture of fertilizers of the variety using the fixation of atmospheric nitrogen.

In Table 1 is given the population of the principal cities of Alabama, showing their growth between 1890 and 1910.

Initial Development.—In July, 1912, it was decided that the initial development of the Company should be at Lock 12 on the Coosa River, rather than at Cherokee Bluffs on the Tallapoosa. This decision was based on a number of considerations, principal among which was the fact that in the Government grant of the Lock 12 site the right to develop power was contingent on the completion of the construction of the dam at this point on March 4th, 1914. If construction had not been prosecuted promptly at this site, a grant of great value to the Company would have been lost. The requisite funds for the initial development had been raised, and each day's

TABLE 1.—POPULATION OF CITIES IN ALABAMA.

	Cities.	Population in Census of: 1890.	1900.	1910.
Northern Alabama.	Tuscumbia	2 491	2 348	3 324
	Florence	6 012	6 478	6 689
	Sheffield	2 731	3 333	4 865
	Decatur	2 765	3 114	4 228
	New Decatur	3 565	4 437	6 118
	Gadsden	2 901	4 282	10 557
	Alabama City	.....	2 276	4 313
	Attalla	1 254	1 692	2 513
	Anniston	9 998	9 695	12 794
	Huntsville	7 995	8 068	7 611
	Birmingham	26 178	38 415	*132 685
	Bessemer	4 544	6 358	10 864
	Tuscaloosa	4 215	5 094	8 407
	Talladega	2 063	5 056	5 854
Southern Alabama.	Montgomery	21 883	30 346	38 136
	Prattville	724	1 929	2 222
	Selma	7 622	8 703	13 649
	Greenville	2 806	3 162	3 377
	Andalusia	270	551	2 480
	Mobile	31 076	38 469	51 521

*In 1909 Birmingham absorbed a number of adjoining towns into the "Greater Birmingham". This accounts for the great increase in population; the rate of increase, however, is remarkably large, the population in 1914 being reported unofficially as 166 000.

delay was costing a large sum of money, and furthermore, it would have been impossible, on account of certain legal difficulties, to start construction at Cherokee Bluffs for a number of months. Besides these facts, the original outlay for the Cherokee Bluffs development was estimated to be more than 50% greater, and the cost per installed horse-power somewhat more, than at Lock 12.

A description of the Coosa River, and of the climatic conditions prevailing in the district, will be given, and, as a large proportion of the membership of this Society is resident in and about New York City, comparisons are made with conditions prevailing in that vicinity.

Coosa River.—The Coosa River, which constitutes part of the drainage system of the Mobile Basin, is formed near Rome, Ga., by the confluence of the Etowah and Oostanaula, which have their sources in the mountains of Northwestern Georgia.

The river flows in a general westerly direction over the Piedmont Plateau from Rome into the State of Alabama, where it turns in a southwesterly direction and meanders through wide valleys to Gads-

den. Here, again, the river changes its general trend, turning more toward the south and flowing in a more direct line, between mountains of limestone formations. A short distance below the crossing of the Louisville and Nashville Railroad, about opposite Sylacauga, the river passes out of the limestone formations into the slates and schists, and the bed of the stream becomes hard and very rough. From this point the river flows in a southeasterly direction over a succession of shoals and riffles to Wetumpka, where it joins the Tallapoosa River to form the Alabama. In this last section, where the fall in the river is more abrupt, a large quantity of power can be developed economically.

The first project for improving the river for navigation was proposed in 1875, and some work has been done in the general scheme of making the river navigable for light-draft boats. The system, when completed, will be a waterway, almost 900 miles long, from the head-waters of the Coosa to the Gulf of Mexico. Prior to 1912, however, no improvements had been made in the rough section of the river between Gadsden and Wetumpka.

In its upper reaches the Coosa flows through clay-covered territory where it accumulates from its feeders, in all except extreme low-water seasons, a quantity of silt, making it appear very muddy. Tests, however, have shown that this extremely muddy appearance is due in great part to material which has gone into solution in the water, as at its worst it carries in suspension not more than 1 part of silt in 6 000. It is not anticipated, therefore, that there will be any considerable difficulty from silting of reservoirs on this river.

The source of the river is in a region which has a heavier annual rainfall than any section of the United States, with the exception of some parts of the State of Washington. The surface rocks are mostly limestones, which, of course, are only moderately porous; but they are broken up with large fissures, and the solvent action of the rainfall has converted many of these fissures into subterranean channels, as shown by the number of large springs which abound in this territory. To a certain extent, this condition, has a regulating effect on the watercourses emptying into the Coosa, yet in the winter and early spring of the normal year the rains are intense, and the river is subject to floods from December until April, with occasional flashy floods before and after that period, the month of greatest flow being March. The regulating effect of these springs and the fact that in

the average year the rainfall is distributed somewhat uniformly over the months are responsible for the fact that the low-water flow of the Coosa is ordinarily nearly three times as great as that of the Hudson at Albany, and is equal to the low-water discharge of the Mississippi at St. Paul. Table 2 gives the discharge record of the river from 1897 to 1914, inclusive.

The greatest flood known in the river occurred in the spring of 1886, before gauge readings had been commenced. At Lock 12 this flood has been variously estimated, by engineers who have studied and reported on the Coosa River projects, to have been from 95 000 to 170 000 sec-ft. Extensive studies made during 1913 have indicated that the probable flood intensity was approximately 135 000 sec-ft. The average spring flood in the last 14 years, during which time observations have been taken continuously has amounted to 80 000 sec-ft.

The area of the water-shed above Lock 12 is 9 087 sq. miles. Using the estimated maximum flood of 1886, 135 000 sec-ft., this would give a maximum discharge of 14.8 sec-ft. per sq. mile of water-shed, which is one of the smallest in the country. This is explained principally by the fact that the water-shed is very large, and the average intensity of rainfall during a storm is very small. To illustrate this, a few examples of flood discharges are given in Table 3.

The lowest flow known to have occurred was in 1904, at which time observations were taken at Riverside, 72 miles above Lock 12. A comparison of the gauges and rating curves at these two points indicates that the minimum flow was 1 700 sec-ft. at Lock 12. The average minimum flow since observations have been taken amounts to 5 000 sec-ft. With the regulation effected by the construction of the dam at Lock 12, the minimum flow of 1904 would have amounted to 2 400 sec-ft.

Climatic Conditions.—Few people in the North realize that Birmingham, Ala., has had a greater range of temperature, both at the upper and lower limits, than New York City. The records of the United States Weather Bureau show that the maximum temperature of New York City is 100° and that the minimum is — 6°, and that the temperature has ranged between 104 and — 10° at Birmingham.

In summer the heat is intense, but of an entirely different character from that of New York City. For long periods the temperature during the day ranges between 90° and 100°, but heat prostrations

TABLE 2.—DISCHARGE OF THE COOSA RIVER

Year.	January.		February.		March.		April.		May.		June.	
	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
1897 to 1913, inclusive.	19 081	94 150	29 478	106 200	35 447	125 700	25 771	112 500	14 184	71 500	18 107	85 500
1897	10 800	30 800	25 100	42 250	55 000	91 000	23 500	66 500	9 150	13 300	6 500	7 250
1898	15 500	55 000	8 400	22 800	8 600	20 200	25 100	56 000	7 200	13 000	5 800	8 200
1899	14 000	22 300	46 750	83 250	61 500	107 250	35 000	67 750	10 300	17 750	7 250	10 300
1900	17 200	49 750	32 500	91 000	38 300	72 000	43 700	112 500	10 800	19 250	30 500	85 500
1901	36 600	94 150	29 650	65 300	28 000	95 700	45 300	84 800	21 150	64 550	15 800	36 900
1902	32 900	94 150	34 600	85 500	53 000	106 500	28 250	92 500	8 750	11 000	6 750	7 400
1903	14 200	26 300	62 150	106 200	57 600	89 800	37 250	79 000	13 350	40 000	18 500	50 200
1904	7 750	18 250	11 000	23 200	14 250	27 750	10 000	18 700	6 400	8 400	6 300	9 650
1905	18 300	73 100	39 000	87 100	14 700	26 800	9 800	11 750	18 500	53 750	8 750	18 000
1906	30 300	64 600	11 200	20 600	50 500	125 700	19 000	56 000	9 300	14 150	13 150	46 000
1907	20 200	70 000	26 000	61 500	26 150	80 800	13 700	24 100	20 650	57 600	14 000	40 750
1908	23 700	53 000	43 200	83 500	27 000	74 650	20 500	37 250	13 300	18 700	8 500	12 200
1909	19 250	46 750	43 750	81 750	70 000	124 000	25 150	63 800	28 300	68 500	33 750	77 800
1910	10 800	21 750	15 850	49 100	15 000	40 800	8 500	17 750	24 000	71 500	15 900	28 750
1911	17 200	67 000	14 150	29 400	9 750	16 800	33 200	84 750	8 750	13 200	6 250	7 250
1912	16 400	40 024	28 350	52 380	37 800	59 620	34 380	61 770	17 050	39 172	12 020	29 800
1913	27 258	89 000	40 968	64 500	60 845	82 560	22 802	55 700	9 772	17 260	8 237	12 280

are practically unknown. The evenings are cool, and the breezes which blow during the night are refreshing after the heat of the day. In its effect on the speed of the work, and consequently its cost, this heat is to be reckoned with, because labor automatically adapts itself to the climate.

TABLE 3.—FLOOD DISCHARGE OF SEVERAL RIVERS.

Stream.	At	Water-shed, in square miles.	Cubic feet per second per square mile.
Six-Mile Creek	Ithaca, N. Y.	47.5	170
Gallinas River	Las Vegas, N. Mex.	90.0	129
Tohickon Creek	Mt. Pleasant, Pa	102.0	112
Nashua River	Massachusetts	109.0	104
Mora River	La Cueva, N. Mex.	159.0	140
Croton River	Croton Dam, N. Y.	339.0	74
Broad River	Carlton, Ga.	762.0	38
Raritan River	Bound Brook, N. J.	879.0	59
Mohawk River	Little Falls, N. Y.	1 306.0	22
Tallapoosa River	Milstead, Ala	3 840.0	18
Hudson River	Mechanicsville, N. Y.	4 500.0	15
Kanawha River	Charlestown, W. Va.	8 900.0	13
Coosa River	Lock 12, Ala	9 087.0	15
Tennessee River	Chattanooga, Tenn	21 418.0	21
Susquehanna River	Harrisburg, Pa.	24 030.0	19
Mississippi River	St. Paul, Minn.	36 085.0	20
Kansas River	Lecompton, Kans	58 550.0	4

The significance of the low limit of temperature reached at Birmingham is not as serious as the records would seem to indicate. Periods of low temperature are very rare, and have never been of

AT LOCK 12, FROM 1897 TO 1913, INCLUSIVE.

July.		August.		September.		October.		November.		December.	
Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
11 991	64 500	10 312	72 800	8 128	62 250	8 895	95 000	7 944	72 400	14 130	96 400
9 250	35 250	6 300	8 400	5 200	6 100	5 050	6 250	5 000	5 550	8 750	17 500
6 800	18 400	12 000	26 800	16 800	62 250	24 000	95 000	10 300	28 700	9 400	13 400
7 800	18 750	6 400	13 200	6 200	9 000	5 450	5 800	5 800	9 650	13 750	36 900
17 500	62 250	7 600	16 250	9 000	31 800	9 500	31 800	10 300	43 800	19 000	30 600
9 000	13 800	27 500	72 800	13 000	31 800	8 150	13 350	6 700	7 150	25 250	96 400
6 250	9 200	6 100	7 900	6 500	13 300	7 000	13 300	6 600	13 000	16 300	33 800
9 950	26 250	8 000	17 500	5 500	7 000	5 600	6 000	5 500	6 700	5 400	5 800
6 000	8 400	8 600	24 250	5 100	5 600	4 500	4 800	4 950	5 100	7 650	18 750
10 700	24 600	9 300	18 700	6 000	7 900	6 600	10 000	5 750	5 800	28 750	56 800
28 700	72 400	14 500	22 900	13 550	19 200	27 200	69 250	23 200	72 400	15 900	46 000
8 450	11 750	7 500	9 650	8 800	29 300	6 700	13 300	12 200	44 500	46 800	38 000
7 500	15 000	7 800	12 150	6 300	13 750	5 600	9 500	5 700	6 300	20 000	63 750
13 750	28 750	15 000	50 500	7 150	11 400	6 750	18 700	5 750	5 800	10 000	20 700
27 000	64 500	8 800	17 750	7 500	15 400	6 000	7 190	5 500	5 600	7 790	24 500
8 550	15 900	9 300	19 200	5 300	5 600	6 700	19 200	8 000	19 650	16 250	56 000
14 650	29 250	10 290	32 000	8 657	16 100	7 524	13 300	5 860	7 320	11 136	19 600
6 752	12 700	6 760	10 500	4 403	17 400	5 635	21 600	3 315	4 000	6 774	14 100

long duration. Ice is never formed to any thickness in ponds or rivers, and the mid-day sun almost invariably causes the temperature of the coldest days to rise above freezing.

It is not necessary, therefore, to consider the effect of an ice thrust in the design of a dam in this climate, and, for the same reason, no precaution need be taken in the design of forebays, for the care and handling of ice at penstock screens. Needless to say, no difficulties are encountered with frazil.

To make clearer the rainfall conditions, Table 4 shows a comparison between the records of normal monthly precipitation at New York City and at Birmingham. From this it will be seen that the rainfall in New York City is more evenly distributed throughout the year than in Birmingham, and that, from December to March, inclusive, the Birmingham rainfall is far in excess of the mean monthly. In July and August, the precipitation at each place is about equal and is the result of heavy showers of short duration.

In the normal year, construction work in Alabama is hampered to a large extent in winter by rains of several days' duration. At such times it is impossible to carry on work, because laborers, even if provided with suitable clothing, will not turn out.

Work Done to Date.—In April, 1912, E. A. Yates, Assoc. M. Am. Soc. C. E., was appointed Chief Engineer of the Company, and an en-

gineering and construction force was organized. A party was put in the field at Cherokee Bluffs for the purpose of surveying the flooded area (34 000 acres) of that reservoir. A short time later an additional party was placed in this reservoir, and a large party was put to work on the surveys for the Coosa River reservoirs. The surveys for the Muscle Shoals reservoirs were made in 1913.

TABLE 4.—COMPARISON OF RECORDS OF NORMAL MONTHLY PRECIPITATION AT NEW YORK CITY AND AT BIRMINGHAM, ALA.

Month.	New York City.	Birmingham, Ala.
January	3.79	5.32
February	3.74	4.75
March	4.10	5.76
April	3.30	3.67
May	3.18	3.09
June	3.26	3.88
July	4.54	4.70
August	4.53	4.48
September	3.59	3.50
October	3.71	2.34
November	3.44	3.39
December	3.45	4.60
Year	44.63	49.48
Mean Monthly	3.72	4.12

At the time the decision had been reached that the initial development should be at Lock 12, no plans of the development had been drawn up, with the exception of a general plan and elevation of the proposed structure. Only 1½ years remained for the construction of the dam within the time stipulated in the Government grant, and, therefore, it was impossible to draw up complete plans before proceeding with the construction. It was desirable to let a contract for the construction of the power-house foundations and dam to a contractor having plant and equipment ready to be moved to the work without delay. Such a contract was let, for the construction of the power-house foundations and dam, on a cost-plus-fixed-fee basis, and the direction of the work, approval of selection of plant, and methods of attack were reserved to the Chief Engineer of the Company. It was decided that the construction of the superstructure and the equipment of the power-house should be done by the Company's own force. As soon as this contract was signed, the necessary preparations for starting construction were inaugurated.

An old lumber railroad (shown on Fig. 3), extended through the hills from a point where it had physical connection with the Louisville and Nashville Railroad, about 5 miles south of Calera, to a point about 5 miles from the dam site. It was known as the Clear Creek Railroad. It was not ideal for construction purposes, because, in its length of 18 miles, it had forty-two timber trestles, one switch-back, and many sharp curves, the sharpest being 35° on a 2.5% grade. The shortest, alternate line, however, would have been at least 14 miles long, and would have involved construction from beginning to end, with a large amount of heavy work. It was necessary to transport the heavy plant to the dam site at the earliest possible moment, and time was not available for the construction of such a line,

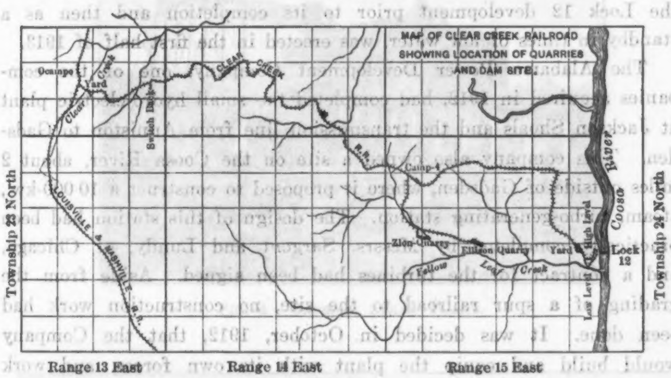

FIG. 3.

although the cost of the railroad as a unit would undoubtedly have been less. The construction and repair work on this railroad was started on August 12th, 1912, by the Company's forces, and on November 26th, the first car loads of the contractor's heavy plant were delivered at the dam site, at least 2 months earlier than would have been possible if an entirely new road had been built. Before this time, however, camps had been constructed at the dam, and preparations were being made for the installation of the plant. During the period when the plant was being set up, a coffer-dam, enclosing the area to be excavated for the power-house and a portion of the spillway, was constructed. This coffer-dam was unwatered for the first time on December 13th, 1912, and the excavation of the river bed was started.

Long delays occurred, due to heavy rains and floods, and it was not until April 12th that the first concrete was placed. In November, 1913, the power-house foundations having been prepared, the building of the superstructure and the installation of machinery were begun. All the concrete work of the power-house foundations and dam was completed on February 26th, 1914, 6 days before the limiting date set by the Government grant. The installation of the first unit was completed on March 22d, 1914. The completion of the installation of the other units followed within a short time thereafter. The first unit was tested out and dried and was put into commercial operation on April 12th, 1914, just 1 year after the first concrete was poured.

An auxiliary steam plant, to be used in building up a load for the Lock 12 development prior to its completion and then as a standby in times of low water, was erected in the first half of 1913.

The Alabama Power Development Company, one of the companies acquired in 1912, had completed the small hydro-electric plant at Jackson Shoals and the transmission line from Anniston to Gadsden. This company also owned a site on the Coosa River, about 2 miles outside of Gadsden, where it proposed to construct a 10 000-kw., steam, turbo-generating station. The design of this station had been practically completed by Messrs. Sargent and Lundy, of Chicago, and a contract for the turbines had been signed. Aside from the grading of a spur railroad to the site, no construction work had been done. It was decided in October, 1912, that the Company would build and equip the plant with its own forces, and work was started under a modified design on November 1st, 1912. The plant was completed in June, 1913, but a delay in the delivery of the transformers prevented commercial delivery of power until July. Fig. 4 shows the exterior of the plant and Fig. 6 shows a section through the boiler and turbine rooms.

The construction of transmission lines and sub-stations was carried on simultaneously with the other work. The surveys for proposed transmission lines were started with three parties in July, 1912, the purchasing of right of way was begun in October, and actual construction was commenced in November. In March, 1914, the lines from Lock 12 to Birmingham (46.5 miles), from Lock 12 to Anniston, *via* Jackson Shoals (47.8 miles), from Sylacauga to Alexander City (25.1 miles), from Jackson Shoals to Leeds (26.5

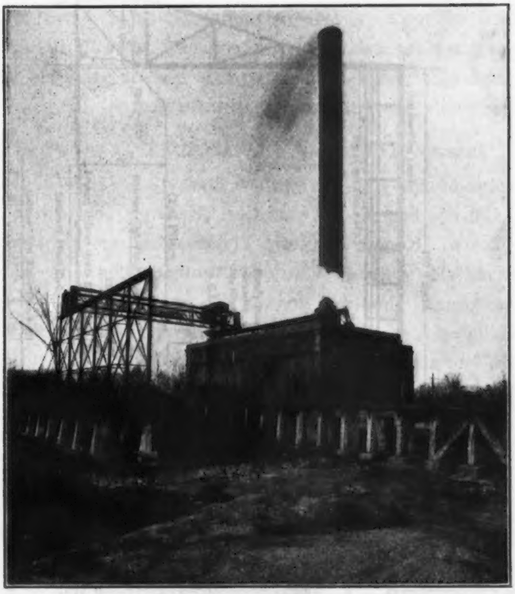

FIG. 4.—GADSDEN STEAM PLANT.

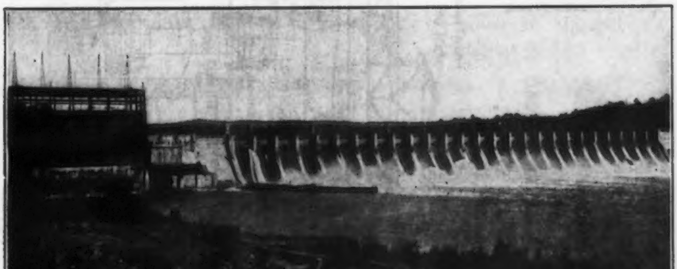

FIG. 5.—POWER-HOUSE UNDER CONSTRUCTION, BRICKWORK HAVING REACHED HIGH-TENSION FLOOR.

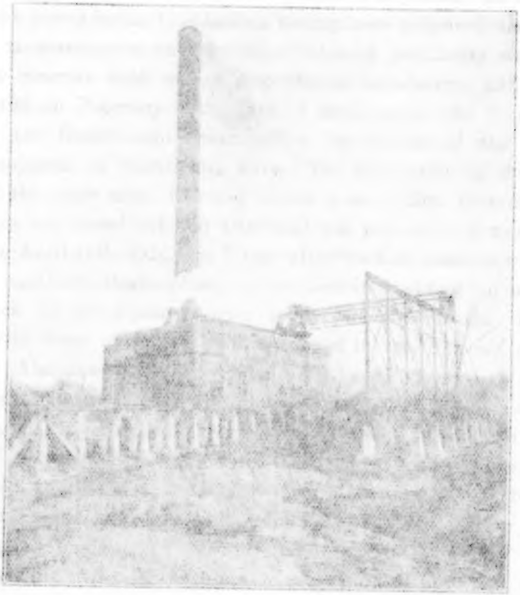

Fig. 4.—Gadsden Steam Plant.

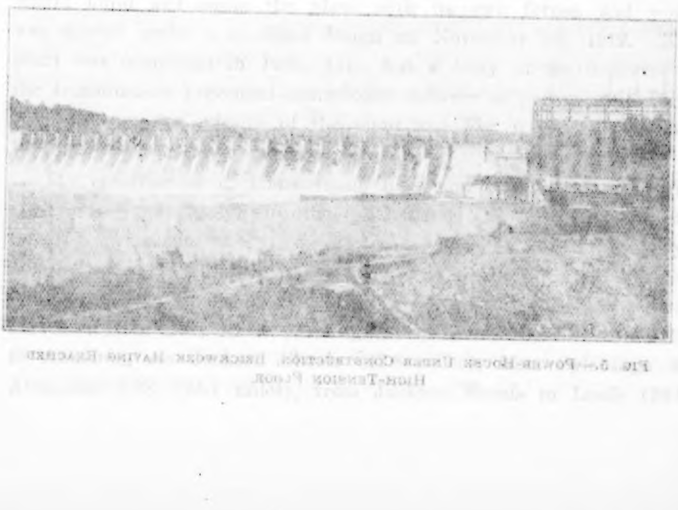

Fig. 5.—Power-House Under Construction. Brickwork Having Reached High-Tension Floor.

CROSS-SECTION OF THE GADSDEN STEAM PLANT

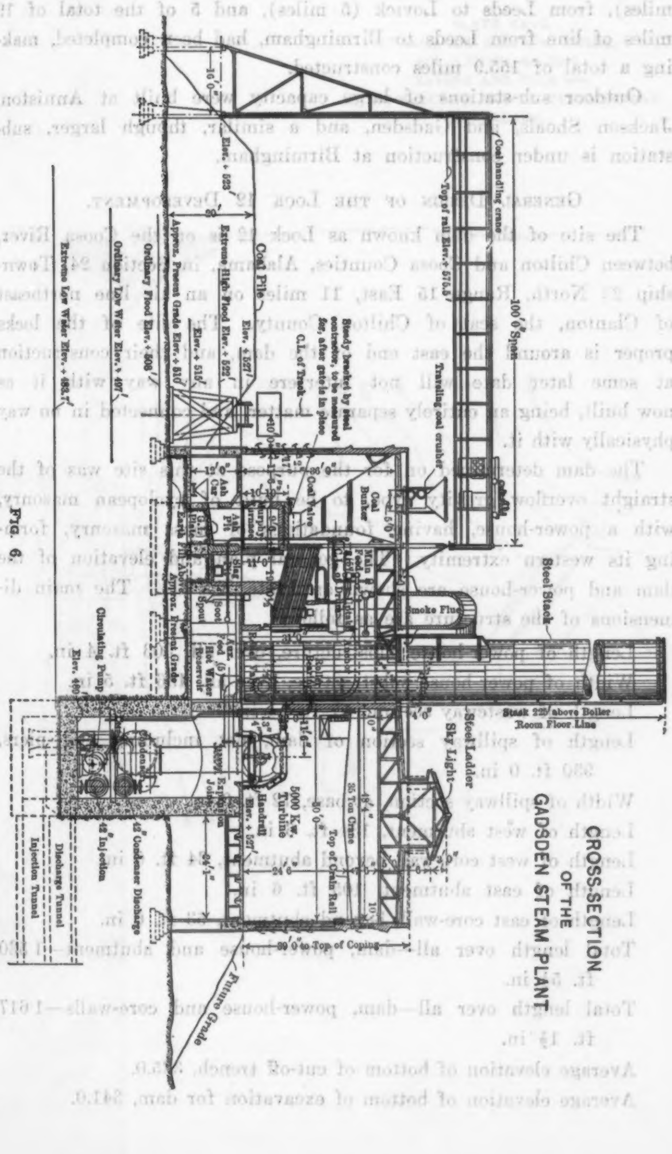

Fig. 6

miles), from Leeds to Lovick (5 miles), and 5 of the total of 19 miles of line from Leeds to Birmingham, had been completed, making a total of 155.9 miles constructed.

Outdoor sub-stations of large capacity were built at Anniston, Jackson Shoals, and Gadsden, and a similar, though larger, substation is under construction at Birmingham.

General Design of the Lock 12 Development.

The site of the dam known as Lock 12 is on the Coosa River, between Chilton and Coosa Counties, Alabama, in Section 24, Township 23 North, Range 15 East, 11 miles on an air line northeast of Clanton, the seat of Chilton County. The site of the locks proper is around the east end of the dam, and their construction at some later date will not interfere in any way with it as now built, being an entirely separate matter, and connected in no way physically with it.

The dam determined on for the purpose at this site was of the straight overflow gravity type, to be built of cyclopean masonry, with a power-house, having foundations of mass masonry, forming its western extremity. The general plan and elevation of the dam and power-house are shown on Plate XXVII. The main dimensions of the structure are as follows:

Length of power-house substructure, over all, 303 ft. 4 in.

Width of power-house substructure, over all, 136 ft. 5 in.

Length of wasteway section, 57 ft. 1½ in.

Length of spillway section of dam, not including end piers, 930 ft. 0 in.

Width of spillway section, at base, 72 ft. 0 in.

Length of west abutment, 134 ft. 6 in.

Length of west core-wall beyond abutment, 34 ft. 6 in.

Length of east abutment, 105 ft. 6 in.

Length of east core-wall, beyond abutment, 53 ft. 0 in.

Total length over all—dam, power-house and abutment—1 530 ft. 5½ in.

Total length over all—dam, power-house and core-walls—1 617 ft. 1½ in.

Average elevation of bottom of cut-off trench, 335.0.

Average elevation of bottom of excavation for dam, 341.0.

PLATE XXVII.
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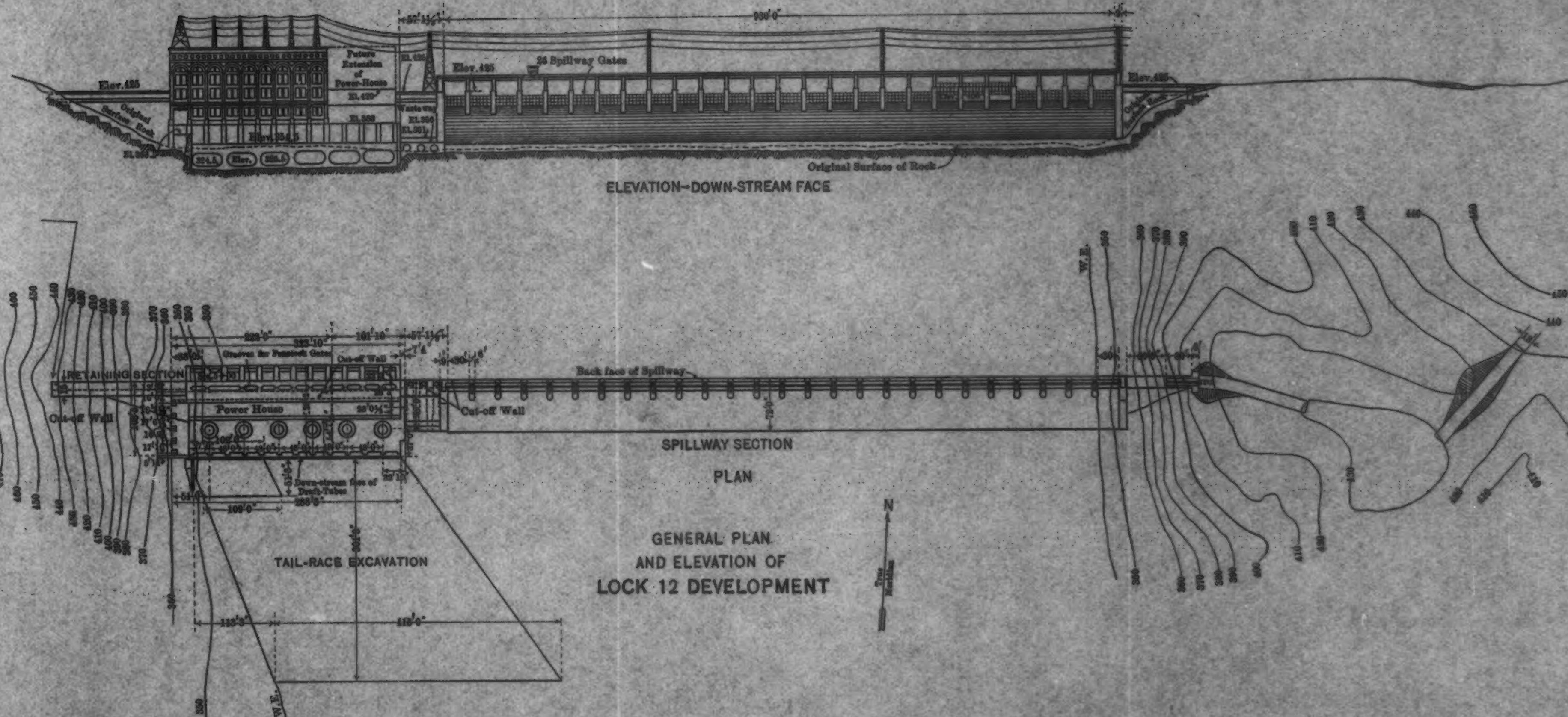
IN ALABAMA.



GENERAL PLAN
AND ELEVATION OF
LOCK 12 DEVELOPMENT

Average elevation of bottom of river before excavation for dam, 343.0.

Elevation of crest of dam, 406.0.

Elevation of top of spillway piers, 444.0.

The spillway, forming practically all of the dam east of the power-house, has a net crest length of 780 ft., divided into 26 sections, each 30 ft. long, which are separated by piers 6 ft. long on the axis of the dam, the piers serving to carry flood-gates of the modified Stoney type.

At a point 11½ miles above Lock 12, the Louisville and Nashville Railroad crosses the Coosa on a through bridge of several trusses, and Peckerwood Creek, a tributary stream, immediately above this point, on a through plate-girder bridge. The elevation of the low steel on the truss bridge is 425.3 and of the plate girders 423.5. These elevations, allowing for the back-water effect which is exaggerated to some extent during floods by the Narrows (shown on Plate XXVIII) about 5 miles below the dam, fix the normal elevation of the pond at 420, as established by the U. S. Army Engineers.

For the purpose of maintaining the normal pond level during floods, twenty-six flood-gates, each 14 ft. high, were designed to be placed on the crest of the spillway at Elevation 406. The abutment sections of the dam and the section between the power-house and the spillway in which are the wasteway culverts, are carried to Elevation 425, with parapet walls to Elevation 429.5.

The power-house is shown in cross-section by Fig. 7, on which it should be noted that the foundations are of mass concrete, the penstocks, scroll casings, and draft-tubes being moulded directly in the concrete with no lining of any kind.

One feature of this development, which is a decided improvement over most of those heretofore constructed, is that the units are entirely self-contained, that is, the turbine, generator, and exciter are self-supporting, and can be built complete, from the base ring of the turbine to the top of the direct-connected exciter, without requiring any lateral support from the concrete about the turbine casing during construction.

There are four units in the power-house, and provision in the foundations for two more. Each of the units has a capacity of

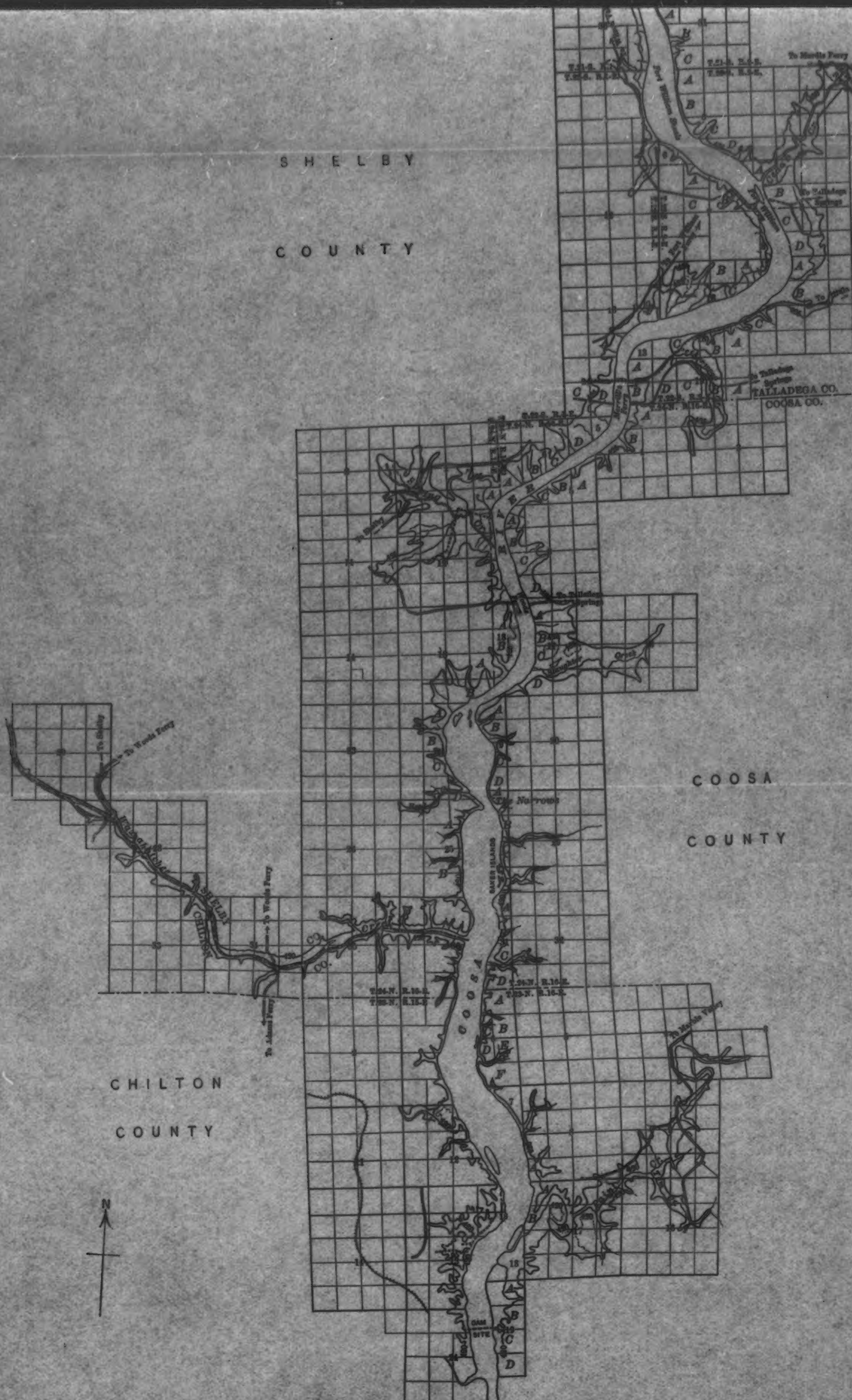
17 500 h.p. delivered to the shaft at a speed of 100 rev. per min., and under a head of 68 ft.

The adoption of this size of installation was dependent on many factors and a number of assumptions, a detailed statement of which is impossible here. It might be well, however, to note that the principal considerations involved were the ordinary working head, the reduction of head at times of flood, the nature and extent of the power market on which the load factor depends, the available steam auxiliary plants, and the relation of the plant at Lock 12 to the other proposed developments of the Company.

As noted previously, the year of lowest flow on record was 1904, when the discharge of the Coosa at Lock 12 dropped to 1 700 sec-ft. The storage of the reservoir, using a 10-ft. draw-down, increases the available draft to 2 300 sec-ft., which, with a 68-ft. head, will produce 14 200 h.p., of continuous, 24-hour power, having 80% efficiency. Combining this with the daily output of a steam plant with a capacity of 15 000 kw., there would be produced 614 000 kw-hr. per day, which, on a 50% load factor, is equal to 51 000 kw. maximum demand. A study of the horse-power percentage of time curve for Lock 12 shows that 70 000 h.p. on a 50% load factor, which is equal to 35 000 h.p., continuous, is available at Lock 12 for 80% of the time under a head of 68 ft. and 85% of the time under a head of 74 ft.

It should be noted that this is a pioneer development in Alabama, and that the character of the market is not as well defined as it is in most States where development has been going on for years. It was not known whether part of the load would be electrochemical, or not, and it was necessary to assume the load factor on the best information obtainable from a hurried survey of the market.

It is estimated that the construction of the Etowah Reservoir at the head-waters of the Coosa, as previously mentioned, will increase the low-water flow of the river to 4 700 sec-ft. Assuming that Lock 12 would be operated ultimately in connection with this storage reservoir and the developments lower down the river, an ultimate installation of 105 000 h.p. is provided for in the power-house foundations.


SHELBY
COUNTY
CHILTON
COUNTY
COOSA
COUNTY
COOSA
RIVER
TALLADEGA CO.
COOSA CO.
SHELBY
CHILTON
CO.
BAKER ISLANDS
The Narrows
To Adams Ferry
To Woods Ferry
To Talladega Springs
To Mardis Ferry
DAM SITE
N

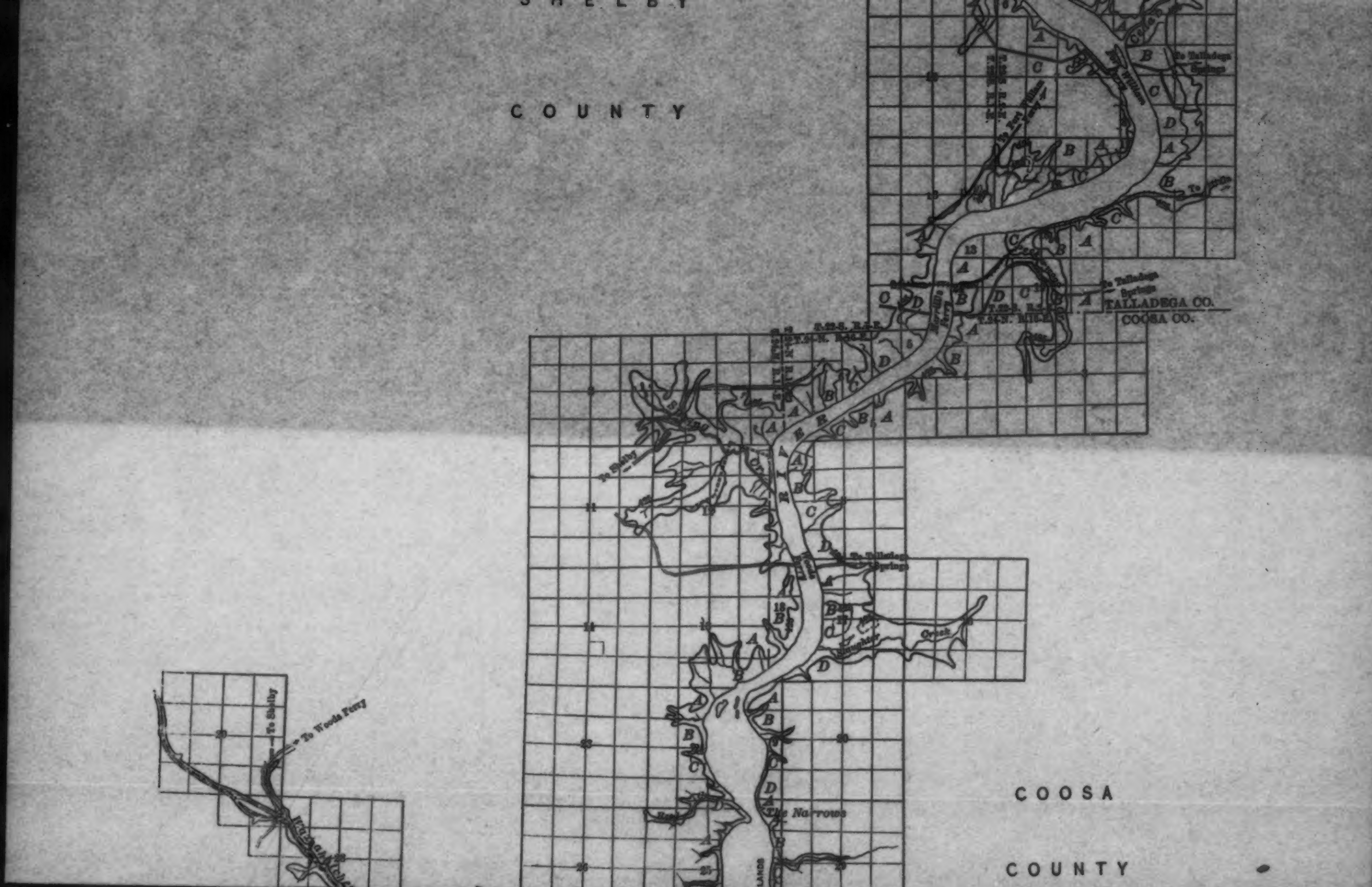

COUNTY
TALLADEGA CO.
COOSA CO.
To Talladega Springs
To Shelby
To Woods Ferry
The Narrows
ISLANDS
RIVER
Creek
COOSA
COUNTY

PLATE XXVIII.
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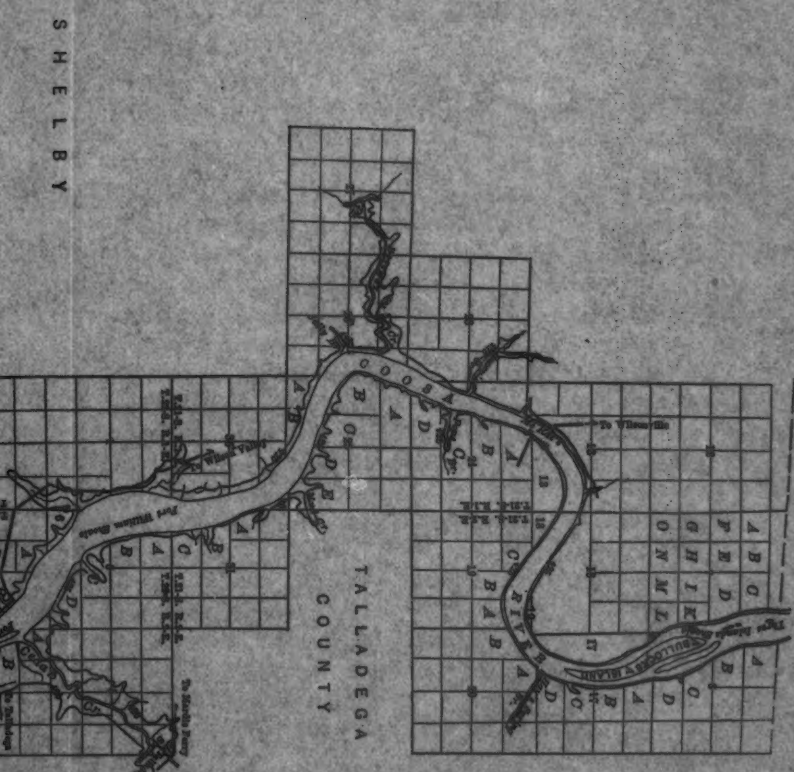
IN ALABAMA.

MAP OF LOCK 12 RESERVOIR



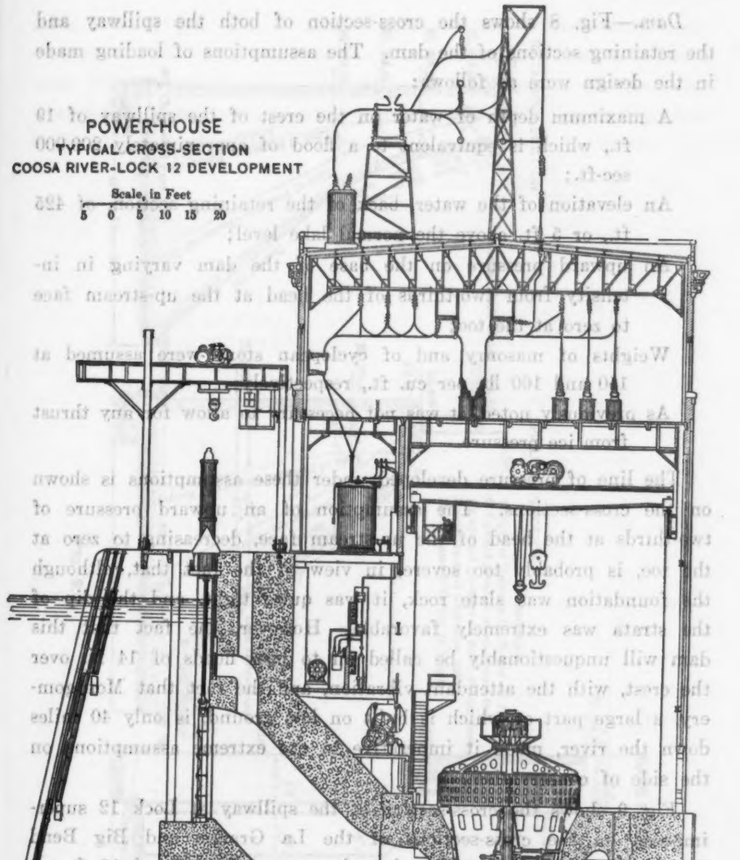

POWER-HOUSE
TYPICAL CROSS-SECTION
COOSA RIVER-LOCK 12 DEVELOPMENT
Scale, in Feet
5 0 5 10 15 20

FIG. 7.

Dam.—Fig. 8 shows the cross-section of both the spillway and the retaining sections of the dam. The assumptions of loading made in the design were as follows:

A maximum depth of water on the crest of the spillway of 19 ft., which is equivalent to a flood of approximately 200 000 sec-ft.;

An elevation of the water, back of the retaining section, of 425 ft., or 5 ft. above the normal lake level;

An upward pressure on the base of the dam varying in intensity from two-thirds of the head at the up-stream face to zero at the toe;

Weights of masonry and of cyclopean stone were assumed at 150 and 160 lb. per cu. ft., respectively;

As previously noted, it was not necessary to allow for any thrust from ice pressure.

The line of pressure developed under these assumptions is shown on the cross-sections. The assumption of an upward pressure of two-thirds at the head of the up-stream face, decreasing to zero at the toe, is probably too severe, in view of the fact that, although the foundation was slate rock, it was quite tight, and the dip of the strata was extremely favorable. However, the fact that this dam will unquestionably be called on to pass floods of 14 ft. over the crest, with the attendant vibration, and the fact that Montgomery, a large part of which is built on low ground, is only 40 miles down the river, made it imperative to use extreme assumptions on the side of conservatism.

Fig. 9 shows the cross-section of the spillway at Lock 12 superimposed on the cross-sections of the La Grange and Big Bend Dams. The former is reported to have passed floods of 15 ft. on the crest, and the latter was designed for floods of 20 ft.

Power-House.—Fig. 7 shows a cross-section of the power-house. The foundations throughout were of mass concrete, reinforced with steel rods around the penstocks, scroll casings, and over the draft-tubes. The computation of the stresses in such a structure is very complicated, as not only the ordinary provisions of dam design must be given attention, but the stresses transmitted from one part of the structure to another must be carefully investigated. The large

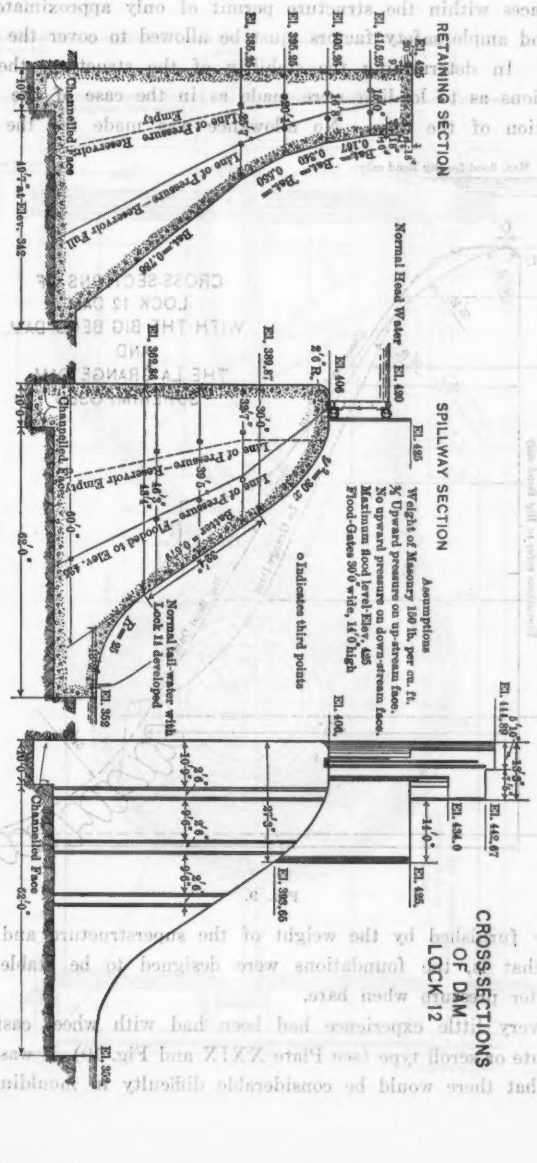

CROSS-SECTIONS OF DAM LOCK 12
RETAINING SECTION
SPILLWAY SECTION
GATE PIER
Assumptions
Weight of Masonry 150 lb. per cu. ft.
⅔ Upward pressure on up-stream face.
No upward pressure on down-stream face.
Maximum flood level-Elev. 435
Flood-Gates 30'0" wide, 14'0" high
o Indicates third points
Normal Head Water El. 420
Normal tail-water with Lock 11 developed
Channelled Face

FIG. 8.

open spaces within the structure permit of only approximate solutions, and ample safety factors must be allowed to cover the uncertainties. In determining the stability of the structure, the same assumptions as to loading were made as in the case of the retaining section of the dam. No allowance was made for the added

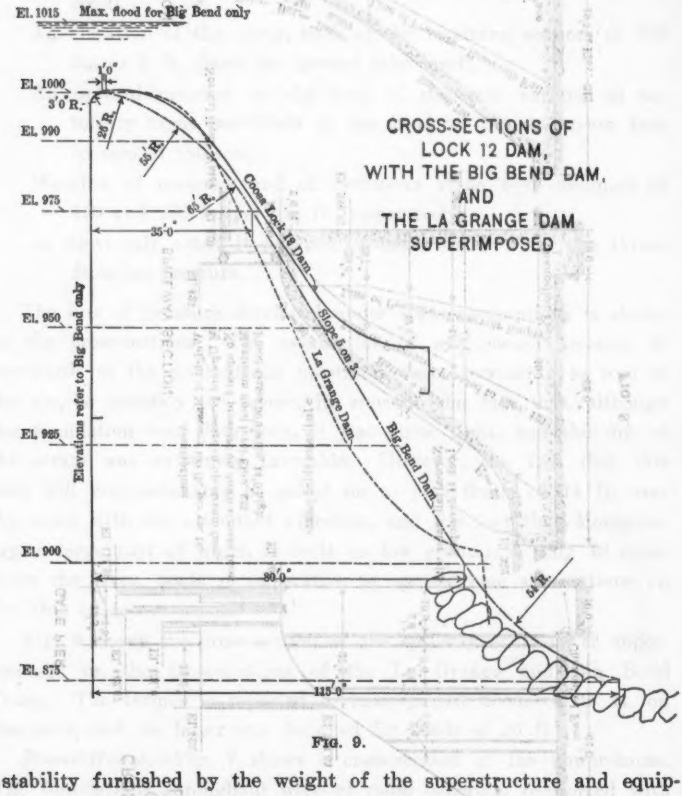


FIG. 9.

stability furnished by the weight of the superstructure and equipment, that is, the foundations were designed to be stable under full water pressure when bare.

As very little experience had been had with wheel casings of the volute or scroll type (see Plate XXIX and Fig. 10), it was anticipated that there would be considerable difficulty in moulding them

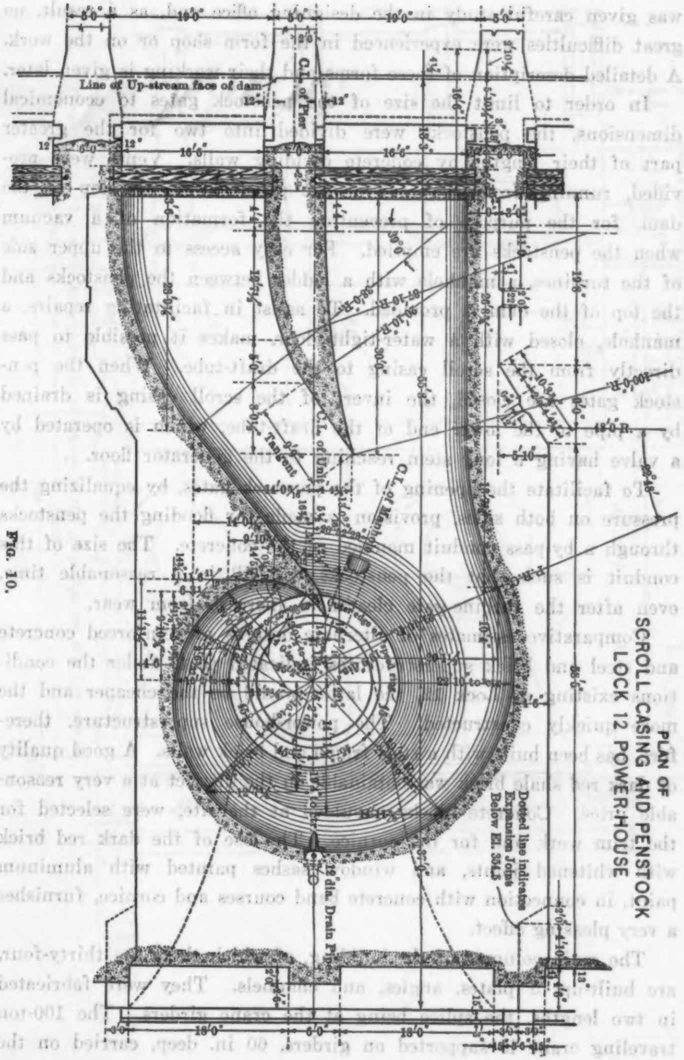
directly in the concrete. For this reason, the structure of the forms was given careful study in the designing office and, as a result, no great difficulties were experienced in the form shop or on the work. A detailed description of these forms and their working is given later.

In order to limit the size of the penstock gates to economical dimensions, the penstocks were divided into two for the greater part of their length by concrete dividing walls. Vents were provided, running [illegible] the [illegible] dam, for the purpose of preventing the formation of a vacuum when the penstocks are emptied. For easy access to the upper side of the turbines, a manhole with a ladder between the penstocks and the top of the turbine is provided. [illegible] in facilitating repairs, a manhole, closed with a water-tight door, makes it possible to pass directly from the scroll casing to the draft-tube. When the penstock gates are closed, the interior of the scroll casing is drained by a pipe at the lower end of the draft-tube, which is operated by a valve having a long stem reaching to the generator floor.

To facilitate the opening of the [illegible] gates, by equalizing the pressure on both sides, provision is made for flooding the penstocks through a by-pass built monolithic in the concrete. The size of this conduit is such [illegible] the [illegible] in a reasonable time, even after the [illegible] wear.

Comparative estimates [illegible] reinforced concrete and steel and [illegible] for the conditions existing [illegible] and the more quickly constructed [illegible] structure, therefore, has been built [illegible]. A good quality of dark red shale brick was [illegible] at a very reasonable price. Concrete [illegible], were selected for the main work [illegible] for [illegible] of the dark red brick with whitened joints, and window sashes painted with aluminum paint, in connection with concrete band courses and cornice, furnishes a very pleasing effect.

The [illegible] thirty-four, are built [illegible] plates, angles, and channels. They were fabricated in two lengths [illegible] of the crane girders. The 100-ton traveling crane is supported on girders, 60 in. deep, carried on the main columns of the building. The roof trusses have a clear span



PLAN OF SCROLL CASING AND PENSTOCK LOCK 12 POWER-HOUSE

Fig. 10.

directly in the concrete. For this reason, the structure of the forms was given careful study in the designing office and, as a result, no great difficulties were experienced in the form shop or on the work. A detailed description of these forms and their working is given later.

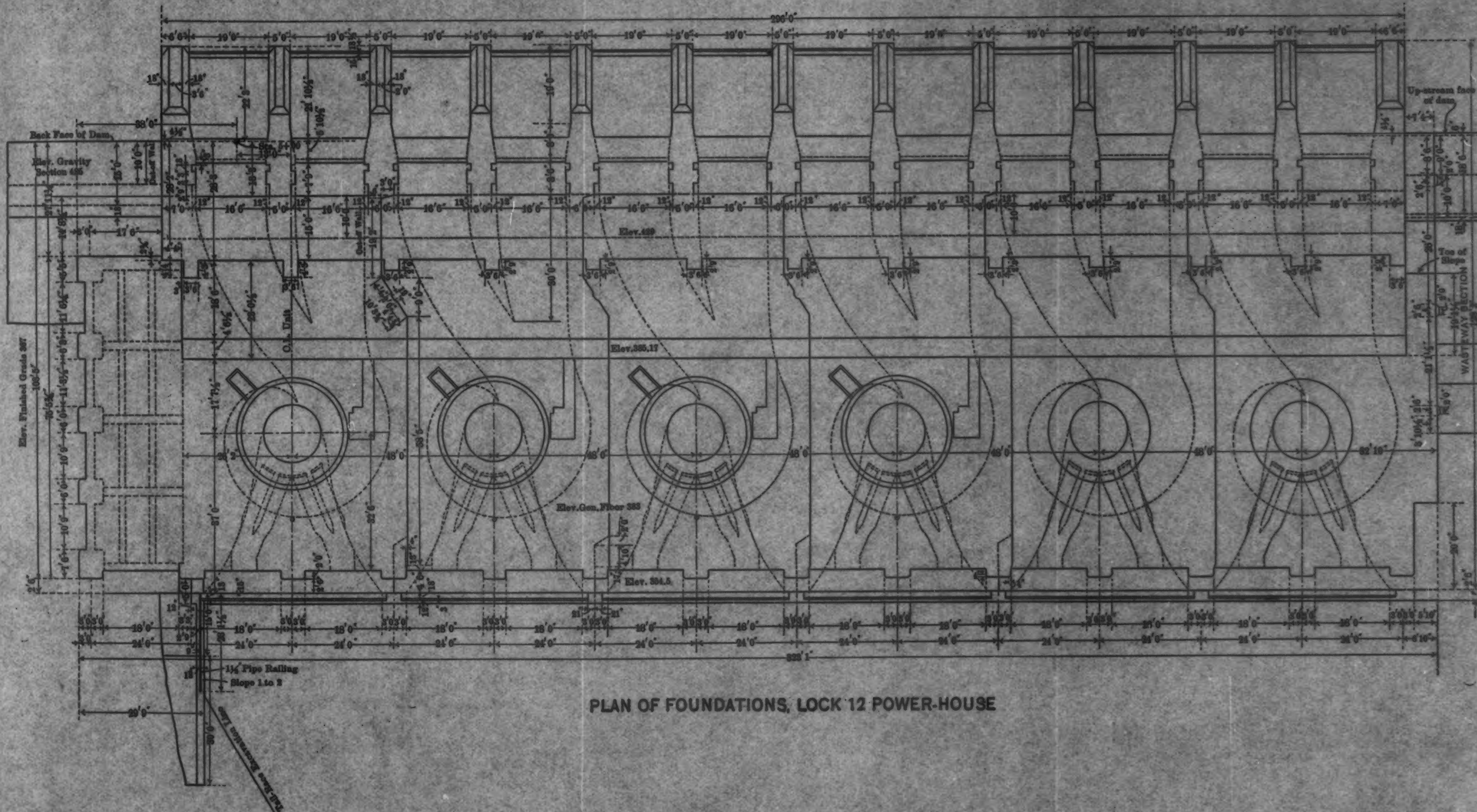
In order to limit the size of the penstock gates to economical dimensions, the penstocks were divided into two for the greater part of their lengths by concrete dividing walls. Vents were provided, running from the roof of the penstocks to the top of the dam, for the purpose of preventing the formation of a vacuum when the penstocks are emptied. For easy access to the upper side of the turbines, a manhole with a ladder between the penstocks and the top of the dam is provided. To assist in facilitating repairs, a manhole, closed with a water-tight door, makes it possible to pass directly from the scroll casing to the draft-tube. When the penstock gates are closed, the invert of the scroll casing is drained by a pipe to the lower end of the draft-tube, which is operated by a valve having a long stem reaching to the generator floor.

To facilitate the opening of the penstock gates, by equalizing the pressure on both sides, provision is made for flooding the penstocks through a by-pass conduit moulded in the concrete. The size of this conduit is such that the penstocks will fill in a reasonable time, even after the turbine-gate clearances increase from wear.

Comparative estimates of alternate designs of reinforced concrete and steel and brick superstructures indicated that, under the conditions existing at Lock 12, the latter would be the cheaper and the more quickly constructed. The power-house superstructure, therefore, has been built with a steel frame and brick walls. A good quality of dark red shale brick was obtainable in the district at a very reasonable price. Concrete blocks, moulded at the site, were selected for the trim work and for the cornice. The use of the dark red brick with whitened joints, and window sashes painted with aluminum paint, in connection with concrete band courses and cornice, furnishes a very pleasing effect.

The main columns of the building, of which there are thirty-four, are built-up of plates, angles, and channels. They were fabricated in two lengths, the splice being at the crane girders. The 100-ton traveling crane is supported on girders, 60 in. deep, carried on the main columns of the building. The roof trusses have a clear span

PLATE XXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1884.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.



PLAN OF FOUNDATIONS, LOCK 12 POWER-HOUSE

of 69 ft., are 9 ft. deep at the mid-point, and, in addition to supporting themselves and the dead load of the roof, are designed to carry, attached to their lower chords, the high-tension bus structure of the power-house, and, on top of the roof, the electrolytic lightning arresters and several outgoing line towers.

The floors of the power-house superstructure are of reinforced concrete slabs supported on steel beams.

Turbines.—The turbines selected for this installation are the largest capacity, vertical, single-runner turbines ever installed in the United States, to operate under a head of approximately 70 ft.

Fig. 11 is a cross-section through a complete unit, and Figs. 12 and 13 show the runner and the pit liner, respectively. The weight of a runner and shaft complete is 103 000 lb., and the weight of a complete turbine is 520 000 lb.

As previously mentioned, one feature in which these turbines differ from most of those heretofore installed is that they are self-contained. The bottom of the pit liner rests directly on the concrete foundation at the scroll-casing level, where anchorages make it possible to adjust the level of any point on it. This arrangement makes it possible to set up the turbine complete in the shop and do all the necessary machine work there, eliminating the very expensive machine work commonly done in the field. The installation of the turbine on its permanent foundation then becomes a comparatively simple matter, involving only the accurate setting of the base ring on the anchorages and the bolting up of the other sections of the pit liner to this ring. If the machine work in the shop has been accurate, the assembly in the field must be simple. The runner of these units is of cast iron and of the Francis inflow type, cast in one piece. The diameter of the runner, over all, is 13 ft. 3 in., and it is 7 ft. 2 in. high.

The casing or pit liner is made up of three rings; a heavy cast-iron ring, in two sections, forms a foundation for the rings above and transmits the weight of the generator and moving parts from them to the concrete foundation; it also serves the purpose of guiding the water from the scroll casing to the movable guide-vanes. This ring of the pit liner is commonly called the speed ring.

Above the speed ring is the intermediate ring. This serves to transmit the load to the speed ring, to form a water-proof lining for the operating chamber over the runner, and to carry the two gate-

operating engines of the turbine. Above this is the third section of the pit liner, the foundation ring, which rises to the elevation of the main floor of the power-house. On this ring is set the generator armature or stator, and the weight of the latter and of all moving parts is transmitted through it. Bolted to this ring, and forming part of it, is

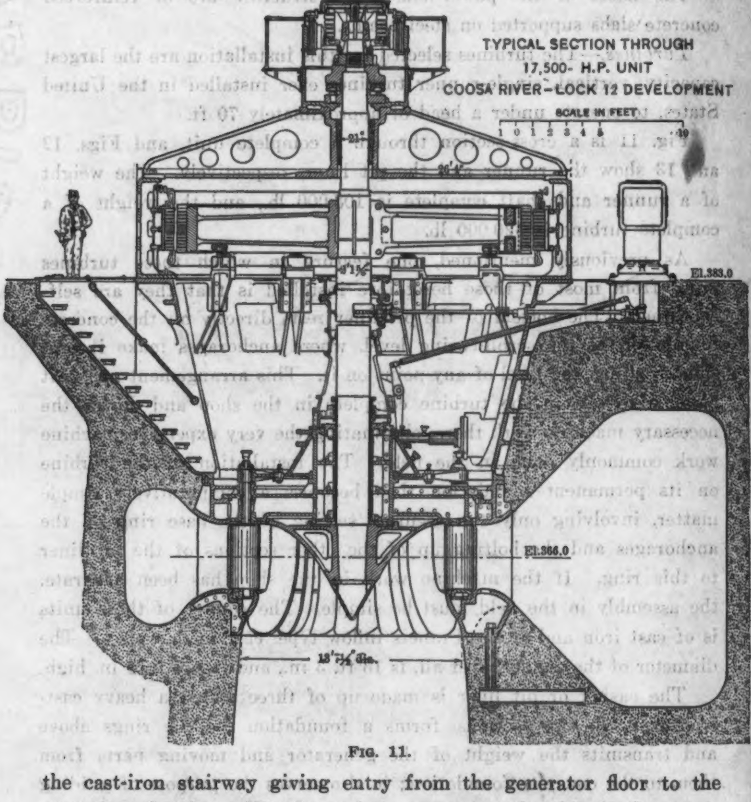


FIG. 11.

the cast-iron stairway giving entry from the generator floor to the operating chamber. The floor of the operating chamber is the cover-head, which is between the speed ring and the intermediate ring of the pit liner.

The shaft, 24 in. in diameter, is one solid piece, passing throughout the turbine, generator, and exciter. The weight of the shaft

Fig. 12.—Runner of Lock No. 12 Turbines in Shop.

Fig. 13.—Complete Pit Liner of Turbine Assembled in Shop.

Fig. 12.—Runner of Lock No. 12 Turbines in Shop.

Fig. 13.—Complete Pit Liner of Turbine Assembled in Shop.

and the running parts attached to it is transmitted through a thrust-bearing, immediately above the generator and below the exciter, to the bridge on the generator. The runner is made a taper fit on the shaft, the torsion being taken on a long vertical key. A circular key and a cast-iron cap hold the runner in place. The shaft is held in position above the runner by lignum-vitæ guide-bearings in the cover-head of the turbine and by a Babbitted steady bearing under the bridge of the generator. The former is lubricated with water, and the latter with oil. The length of contact of the lignum-vitæ guide-bearings is 70½ in.

The turbines in the plant are guaranteed by the manufacturers to deliver at the shaft 17 500 h.p., with a head of 68 ft. and a speed of 100 rev. per min.

The runner selected was stepped up from a model which, on a Holyoke test, developed a maximum efficiency of 89.68%, and the manufacturers have guaranteed that the units as installed will have a maximum efficiency of 87% when delivering to the shaft from 15 000 to 17 000 h.p. The efficiencies guaranteed at full and part gate are as follows:

Power.	Efficiency.
17 500 h.p.	83 per cent.
15 300 "	85½ "
13 125 "	83 "
8 750 "	74 "

The efficiencies obtained in the Holyoke tests are higher than have ever been attained on any turbine heretofore constructed, regardless of speed, capacity, or head.

No opportunity to check the actual efficiencies of the installed units will present itself until the next low-water season, in the summer or fall of 1914, and it is to be hoped that those in charge of the operation of the plant at that time will present a full account of the tests, as they should prove of great interest to the Profession.

These questions received much study, in connection with this development, and it was decided to use vertical units principally because it is possible to obtain higher efficiencies with large capacity vertical than with horizontal units. The reason for this is that in the draft-chests of large horizontal units there are of necessity sharp bends

causing losses of head, and there are other losses from the confluence of streams in the draft-tubes of multi-runner horizontal units. The ideal draft-tube is obtainable with the single-runner, vertical turbine, in which the water is received from the runner at high velocity and conducted away with gradually decreasing velocity to the tail-race, with small loss of effective head from friction or other causes. Volute or scroll casings which, in the last few years, experience has shown to return the highest efficiencies, are impossible to obtain economically in large capacity horizontal units. Besides these considerations, there are other objections to horizontal units in large capacity low-head installations, such as the facts that the unit would be below the level of tail-water, under flood conditions the power-house required would have to be larger, and the structural features of the horizontal shafts for these large wheels would not be satisfactory.

With regard to single *versus* multi-runner vertical units, the following points deserve mention: The ideal scroll casing, by which the inflowing water is brought to the runner at a uniform velocity at all points, cannot be obtained with multi-runner turbines. The effect of multi-runner turbines on draft-tubes is to cause either sharp bends, thereby decreasing the efficiency, or, in an effort to avoid this, require the lengthening of the unit to uneconomical proportions.

The single-runner unit makes possible an ideal arrangement of the operating mechanism, by which it is at all times open to inspection in the operating chamber above the runner. Other advantages of this type are that it is possible to lubricate the gate stems with grease cups, and that there are fewer parts to get out of order.

The selection of speed involves a close study of the inter-relations of power, efficiency, and cost per horse-power developed. In general, other things being equal, the cost of turbines per horse-power developed decreases as the speed increases, the horse-power increases as the speed increases, and the efficiencies at part gate fall off greatly as the speed increases.

An illustration of this last consideration is shown by Table 5, which is a comparison of the efficiencies of these runners at different speeds, under a head of 68 ft. for this installation.

For this development the runner developing 18 500 h.p. at a speed of 100 rev. per min. was selected because a great increase in power was obtainable over the lower-speed runner, with only a small sac-

rifice of efficiency under full load, and the half-load efficiency held up well above that of the higher-speed runner. The consideration that the head would decrease in time of flood to 56 ft. argued against the adoption of a runner with a half-load efficiency as small as 73 per cent. It should be noted that these efficiencies and capacities are not the same as those guaranteed for the full-sized runner, but are those developed from Holyoke tests on model runners.

TABLE 5.

	Efficiency at:		
	94.8 rev. per min.	100 rev. per min.	120 rev. per min.
Developing full power	90.6%	89.5%	90.1%
Developing half power	80.0%	77.0%	73.0%
Full power	16 850 h.p.	18 500 h.p.	17 500 h.p.*

* This runner was smaller than either of the other two, which accounts for the lower power than at 100 rev. per min.

Thrust-Bearings.—The function of the thrust-bearing, as previously noted, is to transmit to the bridge of the generator frame the weight of all the rotating parts and, in addition, the water thrust. Prior to 1912, roller and step-bearings were the types in general use in hydraulic turbines. As its name signifies, the roller-bearing depended on a nest of rollers to reduce the friction while transmitting heavy loads in motion. In some forms of step-bearings, the load is carried by hydraulic or oil pressure at the foot of the shaft. A combination of these two types is in use in some plants. In 1912 a bearing of an entirely different design, which previously had been introduced into steam turbine practice, was applied to the hydraulic turbine. In October, 1912—when the selection of thrust-bearings was taken under consideration—there were only thirty of these Kingsbury bearings in use, and they were mostly in steam turbines under unit pressures of from 300 to 500 lb. per sq. in. of bearing surface. One was in use at the McCall's Ferry Plant and one additional had been ordered. Fig. 14 shows this bearing.

The lower casting with the pockets is stationary, and is bolted fast to the bridge of the generator frame. The segmental castings, with the flat Babbitted surface and the circular pocket on the underside, are the bearing blocks or shoes. These bearing blocks are sup-

ported on disks, the upper surface of which is a sphere of large radius, the disks fitting into the pockets under the bearing blocks. A collar, securely fastened to the shaft and accurately machined on the lower surface, rests on the bearing blocks when the unit is completely assembled. As the bearing blocks rest on a spherical surface, they are free to turn slightly as if on a pivot. When ready for operation, the interior parts of the bearing are completely immersed in oil. The fundamental principle of operation of the bearing, as expressed clearly by the inventor, is "perfect automatic lubrication effected by the pivotal support of the stationary bearing blocks or 'shoes', which permits the formation of an oil film, completely separating the sliding surfaces".

This bearing seemed to do away with the objectionable features of the roller-bearing, namely, large friction losses, high initial cost, inaccessibility, and wear of the moving parts.

The bearing placed in June, 1912, in a 10 000-kw. unit at McCall's Ferry, was under a load of 410 000 lb. at 94 rev. per min. (equivalent to a unit pressure of 350 lb. per sq. in. of bearing surface) and was inspected in October, 1912, after 3½ months of steady service. The Babbitted faces of the shoes had been scraped before being placed, and they still showed distinctly the marks of the scraper. The area of contact, as shown by the bright area of the Babbitt, had extended but slightly, showing that the wear had been negligible. A computation made from the rise of temperature of the oil supplied to the bearing indicated a coefficient of friction of 0.0008. The bearing is made accessible for inspection of the Babbitted surfaces by wedges under the shoes, several of which may be removed at once, without lifting the shaft. The bearings, as made for the McCall's Ferry Plant, had an outside diameter of 48 in. The shaft diameter is 21 in.

After a careful consideration of the claims for the Kingsbury bearing and an inspection of that in service at McCall's Ferry, it was decided to adopt it. As constructed, it is 42 in. in outside diameter and supports a load of 330 000 lb.

A duplicate oiling system has been provided for the lubrication of these bearings. There are two tanks, each with a capacity of 838 gal., on the high-tension floor, from which the oil flows by gravity to the casing of the bearing. Provision has been made for the supply of 15 gal. per min. to the bearings, but this quantity will be regulated as

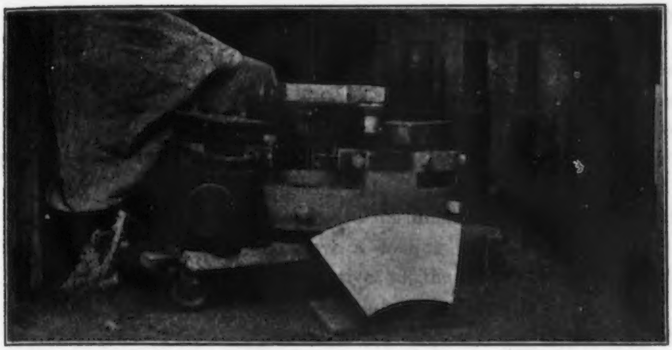


Fig. 14.—Kingsbury Bearing.

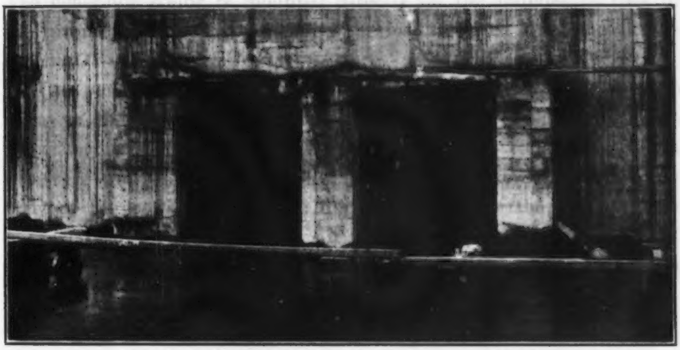


Fig. 15.—Type of Timber Slide-Gates Used to Close Several of the Stream-Control Culverts.

Fig. 16.—Stream-Control Conduits Through Dam, with Flap-Gates in Position for Closing.

experience develops the best method of working the system. The temperature of the oil at the entrance to and exit from the bearing may be noted with the thermometers provided, so that the supply may be regulated. On leaving the bearing, the oil is pumped back through tanks, in which there are cooling coils, to the tanks on the high-tension floor. These coils are 2 ft. in diameter, are made up of 32 turns of 1½-in. pipe, and have a cooling surface of 105 sq. ft. No experience was available relative to the desirable capacity of these coils; their design is based on judgment alone.

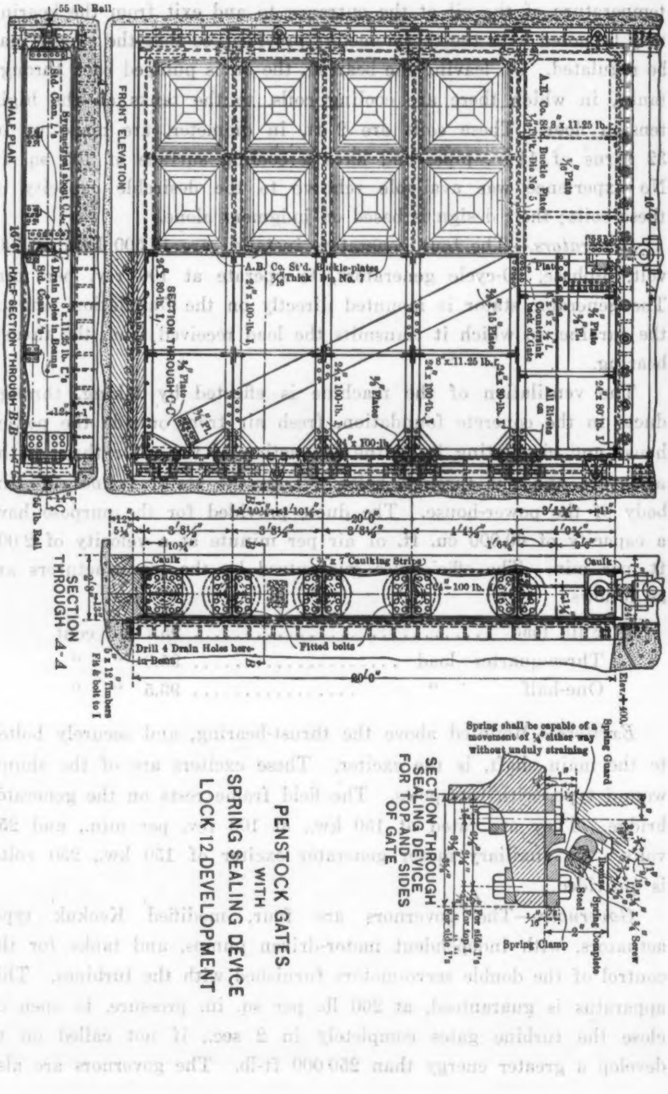
Generators.—The four generators installed are 13 500 kv-a., 6 600-volt, 3-phase, 60-cycle generators, to operate at 100 rev. per min. The generator stator is mounted directly on the foundation ring of the turbine, to which it transmits the load received from the thrust-bearing.

The ventilation of the machine is effected by taking, through ducts in the concrete foundation, fresh air from outside the power-house and circulating it by the fan action of the rotor through the armature and out through holes left in the stator frame into the body of the power-house. The ducts provided for the purpose have a capacity of 60 000 cu. ft. of air per minute at a velocity of 2 000 ft. per min. The efficiencies guaranteed by the manufacturers are as follows:

Full load	95.7 per cent
Three-quarter load	95.0 " "
One-half "	93.5 " "

Exciters.—Mounted above the thrust-bearing, and securely bolted to the main shaft, is the exciter. These exciters are of the shunt-wound type, with interpoles. The field frame rests on the generator bridge. They are rated at 150 kw., at 100 rev. per min., and 250 volts. An auxiliary motor generator exciter of 150 kw., 250 volts, is provided.

Governors.—The governors are four, modified Keokuk type, actuators, with independent motor-driven pumps, and tanks for the control of the double servo-motors furnished with the turbines. This apparatus is guaranteed, at 200 lb. per sq. in. pressure, to open or close the turbine gates completely in 2 sec., if not called on to develop a greater energy than 250 000 ft-lb. The governors are also



PENSTOCK GATES WITH SPRING SEALING DEVICE LOCK 12 DEVELOPMENT

FIG. 17.

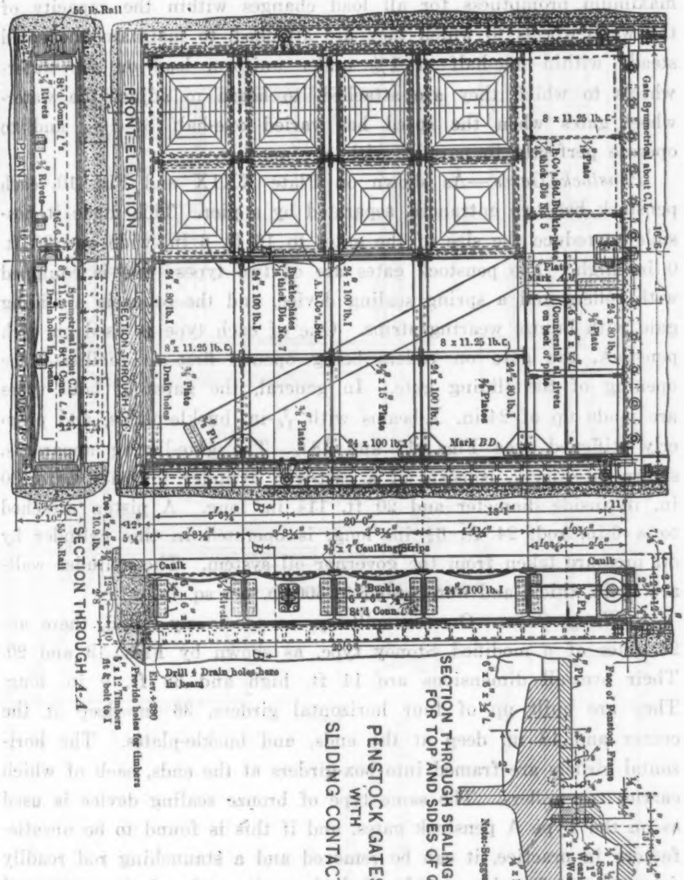

PENSTOCK GATES
WITH
SLIDING CONTACT
SECTION THROUGH SEALING DEVICE
FOR TOP AND SIDES OF GATE
FRONT ELEVATION
SECTION THROUGH C.L.
PLAN
SECTION THROUGH A-A
Face of Penstock Frame
Gate Symmetrical about C.L.
8 x 11.25 lb. C
Mark BD
24 x 100 lb. I
3/8 x 7 Caulking Strips
Caulk
55 lb. Rail
Drill 4 Drain holes here in beam
Provide holes for timbers
Elev. +280

Fig. 18.

guaranteed to stand substantially steady when the speed does not vary, to be dead beat in action, and not to hunt, to correct with maximum promptness for all load changes within the capacity of the water-wheels to which they are attached, to maintain the speed steady within one-half of 1% under uniform load on the water-wheels to which they are attached, to begin to adjust the water-wheel gates when the speed has varied one-half of 1%, and to operate perfectly in parallel with one another.

Penstock Gates.—As shown on Plate XXIX and Fig. 10, each penstock has two entrances, separated by a pier. This made it possible to reduce the size of the gates to 16 ft. 6 in. wide and 20 ft. 0 in. high. The penstock gates are of two types: one is equipped with rollers and a spring sealing device, and the other is a sliding gate with bronze wearing strips. One of each type is used on each penstock, the gate on rollers being opened first to facilitate the opening of the sliding gate. In general, the gates of both types are made up of 24-in. I-beams with $\frac{7}{16}$-in. buckle-plates, and properly stiffened (see Figs. 17 and 18). The gate-lifting apparatus, shown on Fig. 7, consists of a cast-iron cylinder for each gate, 30 in. in inside diameter and 20 ft. $11\frac{1}{2}$ in. long. A piston attached to a 6-in. rod, 24 ft. $6\frac{1}{4}$ in. long, is operated in this cylinder by oil pressure taken from the governor oil system. The cylinder walls are $1\frac{3}{4}$ in. thick, and were tested to 300 lb. per sq. in.

Spillway Gates.—On the spillway, as previously noted, there are 26 gates of a modified Stoney type, as shown by Figs. 19 and 20. Their over-all dimensions are 14 ft. high and 32 ft. 9 in. long. They are built up of four horizontal girders, 36 in. deep at the center and 18 in. deep at the ends, and buckle-plates. The horizontal girders are framed into box-girders at the ends, each of which carries two rollers. The same type of bronze sealing device is used as on the Type A penstock gates, and if this is found to be unsatisfactory in practice, it can be removed and a staunching rod readily substituted. Cast-iron guides, bolted to the ends of the gates and running in cast grooves set in the concrete, prevent the gates from blowing out of line in the wind when in a raised position. The gates are suspended at each end with flat steel ropes passing over drums at the top of the piers. The drums are actuated by shafting and gears, which may be operated from an electric hoist car mounted

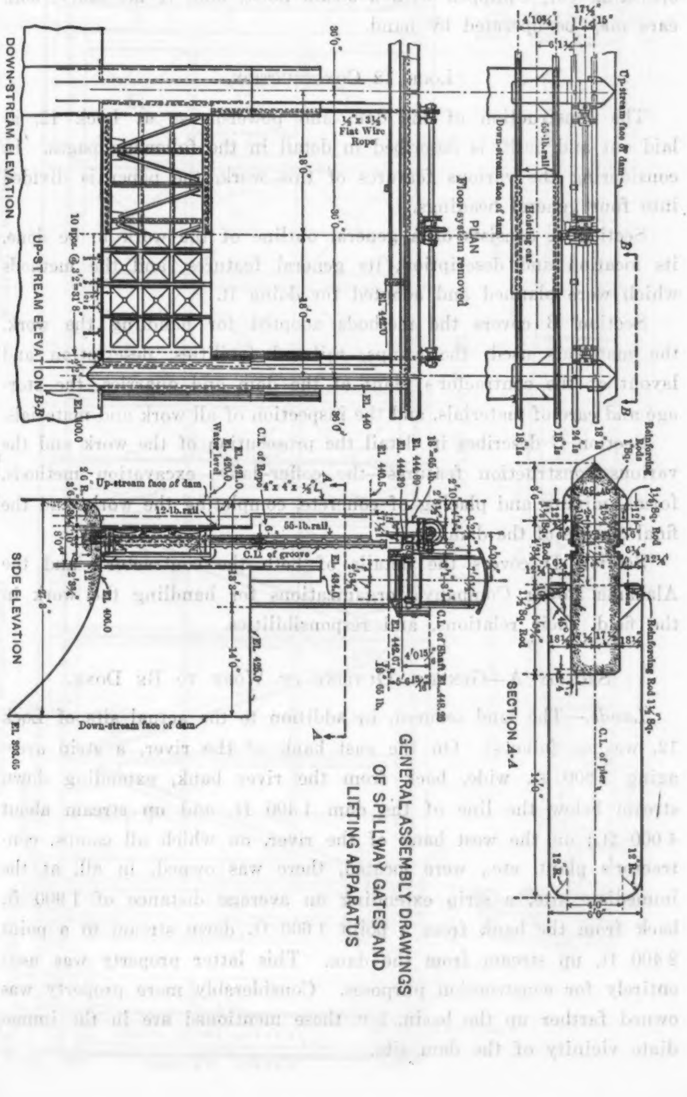


FIG. 19.

GENERAL ASSEMBLY DRAWINGS OF SPILLWAY GATES AND LIFTING APPARATUS

on trucks above the piers. As added security, there is a duplicate operating car, equipped with a steam hoist, and, if necessary, both cars may be operated by hand.

Lock 12 Construction.

The construction of the dam and power-house at Lock 12, as laid out and built, is described in detail in the following pages. In considering the various features of this work, the paper is divided into four general headings.

Section A consists of a general outline of the work to be done, its location and description, its general features, and the methods which were planned and adopted for doing it.

Section B covers the methods adopted for handling the work, the materials used, the camps, railroad facilities, description and layout of the contractor's plant at the dam and quarries, the storage and care of materials, and the inspection of all work and materials.

Section C describes in detail the prosecution of the work and the various construction features—the coffer-dams, excavation methods, forms, mixing and placing of concrete, completing the work, and the final closure of the dam.

Section D covers the details of both the contractor's and the Alabama Power Company's organizations for handling the work in the field, their relations, and responsibilities.

Section A—General Outline of Work to Be Done.

Lands.—The land secured, in addition to the actual site of Lock 12, was as follows: On the east bank of the river, a strip averaging 1 800 ft. wide, back from the river bank, extending down stream below the line of the dam 1 400 ft. and up stream about 4 000 ft.; on the west bank of the river, on which all camps, contractor's plant, etc., were located, there was owned, in all, at the immediate site, a strip extending an average distance of 1 900 ft. back from the bank from a point 1 600 ft. down stream to a point 2 400 ft. up stream from the dam. This latter property was used entirely for construction purposes. Considerably more property was owned farther up the basin, but those mentioned are in the immediate vicinity of the dam site.

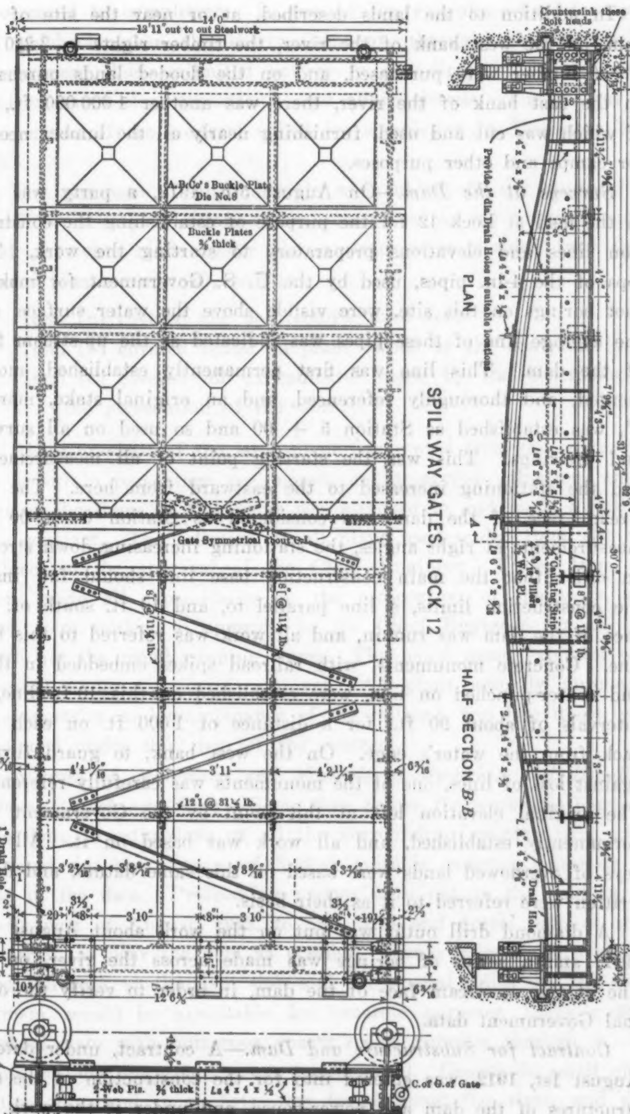

SPILLWAY GATES LOCK 12
PLAN
HALF SECTION B-B
UP-STREAM ELEVATION
DOWN-STREAM ELEVATION
A.B.Co's Buckle Plate Die No.8
Buckle Plates 3/8" thick
Gate Symmetrical about C.L.
13'11" out to out Steelwork
14'0"
Countersink these bolt heads
Provide 1" dia. Drain Holes at suitable locations
Calking Strip
1" Drain Hole
C. of G. of Gate
Separator 3/8" Pl.
2-Web Pls. 3/8" thick

FIG. 20.

sand as was found with it contained such high percentages of clay as to render it unfit for use without thorough washing. Early in the work, therefore, it became evident that sand would have to be secured from outside sources.

The only suitable stone in the district was found, after several months of prospecting, near Zion Church, in Sections 17 and 18, Township 23 N., Range 15 E. (see Fig. 3), and its area was more or less restricted. After carefully prospecting this site at Zion Church and another hill about a mile distant, known later as Ellison Quarry, both sites were purchased. All surface indications and drill borings indicated that Ellison Quarry would be the better of the two; later, however, the contrary turned out to be the case. Both sites were about 11 miles, *via* the construction railroad, from the dam, and were the only available sources of coarse aggregate and cyclopean stone found in the vicinity. Both these hills were heavily overburdened, and, after opening up Ellison Quarry and working it for several months, so many clay seams showed throughout the rock that it had to be abandoned. Although there was plenty of suitable rock available at Zion Quarry, it was only possible to work five derricks, and the stone could not be quarried with sufficient rapidity to satisfy the demands. For this reason, gravel and stone were secured from outside pits to fill out the supply. All the crushing operations were carried on at Zion Quarry, and a general view of the quarry and its layout is shown in Fig. 21.

A chemical analysis of the stone furnished from Zion and Ellison Quarries was as follows:

Aluminum and iron oxide	3.44	per cent.
Carbonate of lime	50.83	" "
Silica	9.00	" "
Carbonate of magnesia	36.38	" "
Combined water and organic matter	0.35	" "

The stone proved to be of excellent quality for concrete purposes, and was far superior to any purchased from outside commercial quarries.

All sand was furnished by contract at a price not to exceed 25 cents per cu. yd., f. o. b. cars, at Jackson's Lake, near Montgomery, Ala. There was a freight charge of approximately 70 cents per cu.

FIG. 21.—CRUSHER PLANT AT ZION QUARRY, SHOWING ALSO PORTION OF STORAGE BIN.

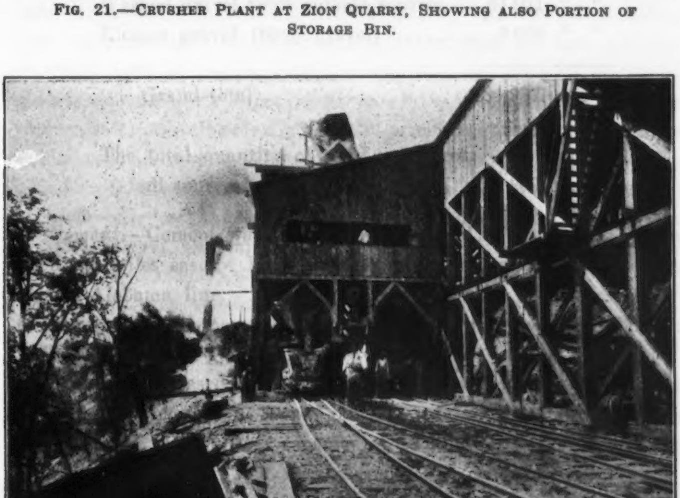


FIG. 22.—CONCRETE-MIXER PLANT.

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The stone proved to be of excellent quality for concrete purposes, and was far superior to any purchased from outside commercial quarries.

All sand was furnished by contract at a price not to exceed 25 cents per cu. yd., f. o. b. cars, at Jackson's Lake, near Montgomery, Ala. There was a freight charge of approximately 70 cents per cu.

Fig. 21.—Crusher Plant at Zion Quarry, Showing also Portion of Storage Bin.

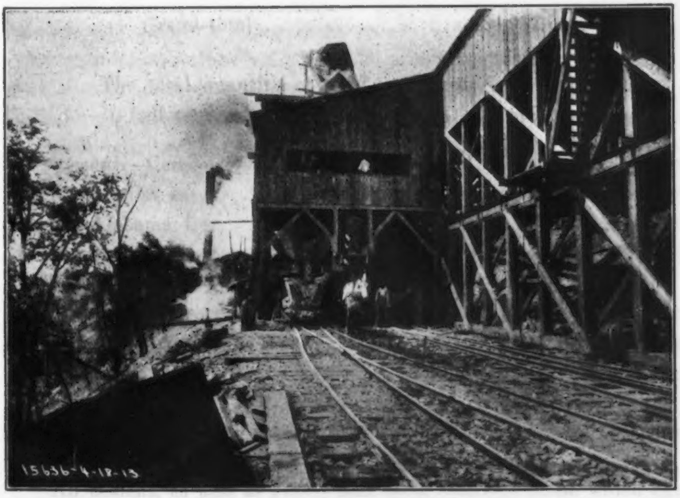


Fig. 22.—Concrete-Mixer Plant.

yd. from Jackson's Lake to Ocampo, making the total cost per cubic yard at Ocampo, 95 cents. This sand proved satisfactory, although certain sections of the pits contained a good deal of loam, and it was necessary for the Power Company to maintain an inspector there constantly to supervise the loading and subject the sand to certain quick field tests for percentages of foreign matter.

All gravel secured from outside sources was washed, with the exception of that brought from the Company's pit at Elmore, Ala., which contained about 45% of gravel and 55% of sand. This pit was purchased to help out at the height of the concreting, and served its purpose.

The following tabulation shows the quantities of materials secured from various sources:

Crushed stone from Ellison Quarry....	11 031 cu. yd.
Crushed stone from Zion Quarry.....	115 207 " "
Cyclopean stone from Zion and Ellison.	7 670 " "
Stone purchased from outside sources.	13 998 " "
Washed gravel from outside sources...	31 031 " "
Elmore gravel (60% gravel).........	6 009 " "
Grand total..........................	184 937 cu. yd.
The total quantity of sand used from all sources was..................	57 809 cu. yd.

Cement.—Cement was purchased from local mills, one in Alabama about 40 miles east of Birmingham and the other in Tennessee close to the Alabama line. It was tested at the mills by representatives of the Power Company, under the specifications contained in *Circular No. 33* of the U. S. Bureau of Standards.

The cement was delivered to Ocampo in carloads of approximately 170 bbl., at a cost of $1.30 per bbl. net. At the dam site a weather-proof storage shed with a capacity of 15 000 bbl. was built and kept full until the work was nearly completed, when the storage was drawn on to tide over periods of car shortage caused by the cotton movement.

All cement, as well as gravel and sand, and that stone which came

from outside quarries, was contracted for by the Company and delivered to the contractor at Ocampo.

Lumber.—Rights to approximately 4 000 000 ft. of timber were acquired in the vicinity of the dam, and contracts were let locally to three small mills to cut it. All these mills were within a mile of the dam, and the sawed lumber was teamed by the Company to the point needed. It is to be noted here that, whenever possible, saw-mills should be on the construction railroad, or should have railroad service.

Timber proved to be very cheap in this section, the price of all the lumber used, of all classes, being about $10 per 1 000 ft. b. m. at the dam.

All the lumber for coffer-dams, camp buildings, bins, shops, stores, forms, etc., was supplied by the above timber and mills. Approximately, 6 600 000 ft. of lumber for all purposes—railroad, dam, and quarries—were used.

Commissary Supplies.—Practically all commissary and other running supplies were secured in the Montgomery or Birmingham market, being bought directly by the contractor. Before the construction railroad was ready for service, all these supplies were brought to Clanton, Ala., 13.6 miles from the dam, by road, and were teamed to the dam by local teams at 20 cents per cwt. Later, when the railroad was in service, a regular merchandise car was run by the Louisville and Nashville Railroad Company from Birmingham to Ocampo and thence taken to the dam on the Clear Creek Railroad. An arrangement was made with the Louisville and Nashville Railroad Company to run a regular package car exclusively for the work at the dam, and during the height of the work, there were two, and sometimes three, cars of merchandise daily to take care of the large force at work.

Coal.—Coal was contracted for by the contractor and delivered in hopper-bottom cars, *via* the Louisville and Nashville Railroad, at Ocampo, from which place it was carried on the construction railroad to the points desired. At the quarry and dam the coal was dumped through trestles directly in front of the boilers where it was to be used. On the railroad it was shoveled by hand out of the cars for engine uses directly into the tenders or to buckets moved by air hoists.

Coal was bought from various mines in Alabama, at a price of \$1.50 per ton, at the mine, and a freight rate of \$1.00, making the price, f. o. b. Ocampo, \$2.50 per ton.

General Scheme of Handling Work at the Dam Site.—The general scheme and outline laid down and followed for the prosecution of the work contemplated the building of three coffer-dams and the handling of the work as follows:

1st.—Coffer-Dam No. 1, which took in the west abutment, power-house, wasteway section, and 404 ft. of the dam, was built and the work within it excavated and concreted. This coffer-dam was designed and built to care for a flow of 75 000 sec-ft. in the river without flooding, and accomplished its purpose. Sufficient openings were left through the portion of the dam constructed within Coffer-Dam No. 1 to care for a flow of 20 000 sec-ft. without flooding Coffer No. 2. The work within Coffer No. 1, it was anticipated, would be accomplished during the high-water stages of the river and that in Coffer No. 2 during the low-water, summer season.

2d.—The building of Coffer No. 2, turning the river through the culverts left in the first portion of the dam.

3d.—The construction of the second portion of the dam and east abutment, leaving more openings through it, with their bottoms at a higher elevation, in order to be able to pass a large flood if the river rose rapidly in the fall before the power-house end of the work was finished, and also making it possible for the reservoir to be drawn down if the water did rise to a considerable height before the work was completed.

4th.—The construction of the tail-race coffer-dam at about the same time as Coffer No. 2, and the excavation of the tail-race while the second section of the dam was being concreted. The power-house concreting was to be prosecuted while the first section of the dam was being built, but it was anticipated that it would be far behind that portion of the work, and would come to completion about the same time as the second section. Consequently, a small concrete wall, from the end of the power-house next to the wasteway section up stream, and a coffer-dam down stream to the tail-race coffer, were built in order that when the water was turned through the first section of the dam, the power-house and tail-race work would still be protected.

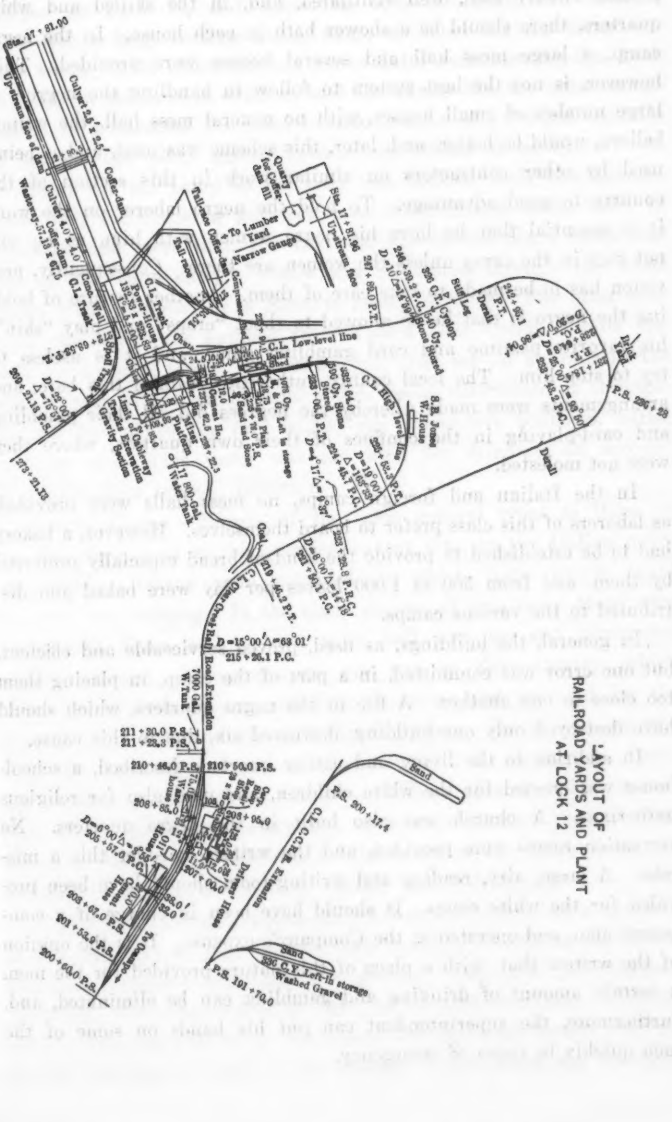
Section B—Methods Adopted for Handling the Work.

Camps.

The very first problem which presented itself in the prosecution of the work was the building of camps, and the providing of necessary messes and quarters for the men and houses for their families, as it was realized that unless there was proper food and comfortable habitations, it would not be possible to hold the men. It was anticipated that a very large force would be necessary in order to complete the work quickly and on time, and that, with such a large force, the sanitary and medical part of the work would need strict attention, and would be no small problem in itself. It was also essential that no contagious or infectious disease be allowed to gain a foothold and sweep through the camps, seriously hampering the work and giving the job a bad name. All camp buildings were constructed of wood, and were fairly well ventilated and cared for.

Lock 12 Camp.—At the dam five distinct sections of the camp were built. The white camp contained the commissary, white mess hall, bunk-houses, foremen's houses and families, and the houses of the contractor's staff, ice plant, bakery, storehouse, etc., in fact, this was the main camp. The engineers' camp was by itself, near the contractor's main camp No. 1, and contained a bunk-house, family dwellings of the staff, a mess hall, office, and a special house for the accommodation of Company guests and visitors, known as the "Guest House". The negro quarters were about ½ mile from the main camp, on one side of the railroad. About 400 yd. from the negro quarters was a fourth camp, for foreigners. The Swedish carpenters lived in a camp by themselves. The camps for the negroes and foreigners were combined into one large negro camp after the foreign laborers at the dam were discharged. Fig. 23 is a general map of the Lock 12 yards, railroad connections, etc.

About 1 000 persons could be accommodated at the main Lock 12 Camp, and at the height of the work it contained about that number. The topography at Lock 12 prevented any general scheme of laying out the buildings; they were placed on the tops of the ridges, with the drainage away from them toward the river and creeks. Where possible, a regular arrangement should be adhered to, and is also recommended from a sanitary standpoint. Some of the bunk-houses at this camp were too large, and did not afford proper ventilation at



LAYOUT OF RAILROAD YARDS AND PLANT AT LOCK 12

FIG. 23.

night. A house of proper size would be one large enough to accommodate twenty men, well ventilated, and, in the skilled and white quarters, there should be a shower bath in each house. In the negro camp, a large mess hall and several houses were provided. This, however, is not the best system to follow in handling the negro. A large number of small houses, with no general mess hall, the writers believe, would be better, and, later, this scheme was used, and is being used by other contractors on similar work in this section of the country to good advantage. To hold the negro laborer on the work it is essential that he have his negro women with him, as he will not stay in the camp unless the women are there. Consequently, provision has to be made to take care of them. Another feature of holding the negro is that he be allowed to shoot "craps" and play "skin", his favorite pastime and card gambling game, and it is useless to try to stop him. The local county authorities realized this here, and arrangements were made whereby the negroes did all their gambling and card-playing in the confines of their own quarters, where they were not molested.

In the Italian and foreign camps, no mess halls were provided, as laborers of this class prefer to board themselves. However, a bakery had to be established to provide the kind of bread especially preferred by them, and from 500 to 1 000 loaves per day were baked and distributed to the various camps.

In general, the buildings, as used, proved serviceable and efficient, but one error was committed, in a part of the camp, in placing them too close to one another. A fire in the negro quarters, which should have destroyed only one building, destroyed six, due to this cause.

In addition to the living and eating quarters described, a schoolhouse was erected for the white children, and used also for religious gatherings. A church was also built in the negro quarters. No recreation rooms were provided, and the writers consider this a mistake. A large, airy, reading and writing-room should have been provided for the white camp. It should have been in charge of a competent man, and operated at the Company's expense. It is the opinion of the writers that, with a place of this nature provided for the men, a certain amount of drinking and gambling can be eliminated, and, furthermore, the superintendent can put his hands on some of the men quickly in times of emergency.

A small $1\frac{1}{2}$-ton ice plant was installed at the rear of the white kitchen at Lock 12, and, adjoining both the plant and kitchen, there was a large cooling room, where all meat for the camp was kept fresh. This plant did not prove quite sufficient for the Lock 12 camp in the hottest weather, and the supply for the other camps had to be contracted for from outside sources and hauled in daily by team. The plant proved satisfactory except as noted.

Water for the camp at the dam was secured from two sources. That used for drinking purposes came from drilled wells, 6 in. in diameter, and more than 130 ft. deep, one in the negro camp and one in the white camp. Water was circulated through the white camps from this well by a gasoline pumping outfit. For other purposes the water was pumped from Yellow Leaf Creek, $\frac{1}{2}$ mile distant, by a steam pump, to a 20 000-gal. tank set on a tower in the highest part of the camp, from which mains distributed it throughout all the camps for fire, boiler, and locomotive purposes, baths, washing, etc. This water was not used for drinking on account of danger from typhoid infection.

Quarry Camps.—Houses of the same general types as those at the dam were used at the quarries, except that, in the Italian quarters, little kitchens were provided in which these people could do their own cooking. In the negro section of this camp there were tents, with floors and wooden sides half way up and permanent wooden frames. A commissary was run at Zion Quarry, which took care of both Ellison and Zion Quarries, and provided for the general mess hall and the individual messes. Bread was secured from the Lock 12 bakery, and ice, to keep the meat, etc., fresh, was hauled by team from Clanton. Ellison Quarry, while operating, had a small commissary run as a subsidiary to that of Zion Quarry. Water was provided at the quarries by wells, from 100 to 150 ft. deep, drilled with a regular well-drilling rig.

Lighting of Camps.—Steam-driven lighting plants were provided at the dam and at Zion Quarry, and furnished lights to all houses, shops, storehouses, etc., and the general camp and job lighting. Dwellings were supplied with light, at an equitable rate, if so desired. Flaming arc lamps were used on the work proper and for general lighting throughout the camps. Later, when the transmis-

sion lines were connected up to Gadsden, the steam plant was shut down at Lock 12, and current was secured from this source.

Hospital, Medical Service, Sanitation.—All medical and hospital service was handled under a contract with a local physician who lived at the dam. Under the contract he was to render services at all the camps, provide all drugs, medicines, a hospital, nurses, etc., and receive in return 90% of the collections from the men, which amounted to $1 per month per man.

The doctor also acted as sanitary officer, but merely reported what he saw and had no authority to enforce his rulings, which was not a desirable feature. Sanitary precautions were taken by providing the pail system of closets and making the boxes of the closets fly-proof. The refuse matter was gathered nightly by a sanitary wagon, hauled a considerable distance from camp, and buried. All closets were kept well sprinkled with lime at all times. All kitchens and dining-rooms were screened, as well as practically all the family dwelling-houses.

The general health of all the camps was remarkably good. No contagious or infectious diseases originated at any of them. At the quarry camp several cases of typhoid fever developed among the Italians, and was traced to some bad meat which they had secured from an outside source. Only about six or eight cases came from this source, one of them being an inspector on the Resident Engineer's staff.

All accidents were treated immediately by the resident physicians, at the camps in which they occurred, and serious cases were brought to Lock 12 to the general hospital, where nurses were maintained.

Policing the Camps.—A regular force of deputies was maintained by the Power Company, regularly commissioned under the Sheriff of Chilton County, there being two deputies at Lock 12, one at the quarry, and one on the railroad. The most troublesome features arose from bootleggers bringing whiskey into the camps.

Camp Bosses.—Camp bosses looked after the cleaning and care of the camps; and their assistants, known as shack rousters, looked after the cleaning of the bunk-houses, etc. In the negro camp, the camp boss was a responsible white man who thoroughly understood the negroes and saw that they were not allowed to lie around

the camp, unless actually ill. He kept track of the various families occupying the small houses, looked after the checking of the dining-room, kept his eye on bad negroes, and, in general, maintained order. He also assigned quarters to new men coming in and looked after them, seeing that they got out to work properly.

Layout of Contractor's Plant and Details of Various Main Parts of the Plant.

Railroad Yards and Tracks.—At Lock 12 extensive yards and tracks for the storage and rapid handling of materials were provided; and a line was built down into the river bottom, below the dam, whereby trains and engines could get down on the cofferdam; this was known as the "Low-Level Line". There were two storage yards for sand and gravel, where loading and unloading operations in no way interfered with regular train and switching movements. A long side track served the main cement warehouse, oil house, and general store-house, so that cars were unloaded directly into them. The shops were also served with another siding, but the arrangement used was a poor one, and detailed criticism of this feature of the layout is given elsewhere. The main line, known after it left the yards on its way to the dam as the "High-Level Line", served the mixer, cement warehouse, concrete material bins, and compressor plant, materials being unloaded directly to these various places from the cars. Fig. 23 is a map showing this detailed track layout.

Cableways.—Two traveling cableways spanned the river parallel to and immediately over the face of the dam. Together, the two cableways had a range of about 150 ft. up and down stream, that is, one could be placed over any portion of a strip 150 ft. wide at right angles to the line of the dam. The towers of these cableways were 95 ft. above the rail, their track elevation being about 440. The length of span, from center to center of towers, was 1 647 ft. The towers traveled on standard-gauge tracks and on standard M.C.B. car wheels and axles. The main cables were $2\frac{1}{4}$ in. in diameter and of patent, lock-laid, flat, steel strands on the outer covering. The cableways were also equipped with one button line, having eight buttons on it, one endless line, and one hoist line. The button and hoist lines were $\frac{3}{4}$-in., $\frac{6}{19}$-strand, Hercules, wire ropes.

The endless line was a $\frac{3}{4}$-in., $\frac{6}{25}$-strand, flat, wire rope. The button line was 1 700 ft. long, the hoist line, 2 100 ft., and the endless line, 2 600 ft. long. The cableways were operated by $12\frac{1}{4}$ by 15-in. engines, driven by steam, supplied from 80-h.p., locomotive-type boilers, all mounted on the trucks of the towers. The cableways had a rated capacity of 10 tons, and were operated under a sag of 80 ft. with a load of 6 tons.

In the early stages of the operation of the cableways, a good deal of trouble was experienced with carriers. The original carriers seemed to be too heavy, frequently breaking each other when they struck. Lighter carriers were made, and much of the trouble was eliminated.

Trouble was also caused by the breaking of two strands in No. 2 cable. The first breaks were welded by the oxy-acetylene process. The weld did not prove satisfactory, and they broke again; this cable was used for the remainder of the job with these strands cut out.

The buttons used on the button lines were of an obsolete type, and were not satisfactory; when one of them broke or slipped along the cable, it was necessary, in order to put on a new one, to lower the cable, cut out a section, and replace it, all of which was expensive and caused much delay. There is now manufactured a cast-steel button, made in two parts, which does away with the necessity for lowering the cable. A button of this type can be put on in an hour, and is a great improvement over the old one. Extra hoist, button, and endless lines were kept in readiness at the head-towers, so that, in case of a breakdown at any time, they could be quickly and rapidly installed. The cableways handled practically 60% of the concrete, and were run both day and night. They also handled practically all the excavation in the early stages of the work. They were used for moving and erecting all derricks, etc., handling form lumber, iron, and materials of a like character, and were found to be exceedingly useful for the latter purpose.

Derricks.—All derricks used at the dam for excavating and placing concrete were of 5-ton capacity, of the wooden, traveling, stiff-leg type, with 37 to 40-ft. masts and 55 to 65-ft. booms. These derricks traveled on the up-stream and down-stream coffer-dams, parallel to the line of the dam, and on the bottom in the tail-race

and draft-tube excavation of the power-house. Later, when the concrete reached Elevation 380.0 in the power-house, three of these derricks were set up on high bents and were stationary.

All the derricks were equipped with $\frac{5}{8}$-in., $\frac{6}{19}$-strand, Hercules, steel-wire rope hoist and boom lines, and with 14-in. derrick irons and 12-ft. bull-wheels. Double blocks were used on the booms and single blocks on the hoist lines, and were operated by 7 by 10-in. engines, with swinging gears, driven by air. The traveling feature of these derricks seems to be an excellent one, as their increased radius of action is most advantageous. In this case the back trucks of these derricks traveled on the top of the coffer-dam, and the front trucks on a trestle resting on the bottom inside. Three of them were able to cover a length of 475 ft. of the dam, on the up-stream face. These derricks handled excavation in skips into cars run out on the up-stream coffer, and took concrete from the cableways, placing it in the forms, the cableways depositing a loaded bucket in a certain place and removing the empty bucket, the derrick then taking the bucket and dumping it in any portion of the form desired. They also took concrete from flat cars run out on the up-stream coffer-dam.

Mixer Plant.—The mixer plant was about 200 ft. south of the cableways, and discharged concrete to cars standing at Elevation 425. The plant consisted of two Model 64, 84-cu. ft. mixers, owned by the Power Company, and driven by an 18 by 24-in., horizontal, single-cylinder engine, rented from the contractor. This engine was run by steam supplied from the compressor plant, about 100 ft. away, as shown in Fig. 22. These mixers had a capacity of 20 batches per hour, their combined capacity being 3 200 cu. yd. for two 10-hour shifts. The highest run made was 2 220 cu. yd. Materials for concrete came in in hopper-bottom cars, and were dumped through a trestle into a bin having a capacity of 800 cu. yd. They were removed from this bin by a belt conveyor to a bucket elevator, and deposited in smaller bins over each mixer, from which they were drawn to the measuring hoppers. The belt conveyor and bucket elevator were driven by a 10 by 16-in., horizontal, single-cylinder, engine, also run by steam supplied from the compressor plant. The drive on both the mixers and elevators was with belts, to line shafts, and friction clutches. One of the drums of the mixers wore through,

comparatively quickly, due to the exceedingly tough and hard quality of the stone. This drum mixed approximately 70 000 cu. yd. of concrete. These drums were of $\frac{1}{4}$-in. metal, and the writers believe this to be somewhat too thin for this size. They suggest that a $\frac{3}{8}$-in. shell would be preferable. Four sets of blades and one drum were worn out on one mixer, and three and one-half sets of blades on the other, in the course of the job.

Two elevators were installed during the progress of the work. The first was a No. 6, 18-in. bucket, stone elevator, 67-ft. centers, on the main pulleys. It did not have sufficient capacity to supply 1 000 cu. yd. per 10-hour day to the mixers, as required, and was taken out in August and replaced with a No. 8, 30-in. bucket elevator. A 32-in., 6-ply belt, 147 ft. long, was used, and had ninety-eight 30 by 17 by 10-in. steel buckets. After the installation of the larger elevator, there was no more trouble.

Water.—Water was supplied to the mixers through a 2-in. line from the main storage tank at the dam. Two small iron tanks with the necessary control valves, gauge glasses, etc., were equipped and set up overhead in the mixer-room, and all the water was drawn directly from these to the mixers.

Empty Sacks.—As soon as the cement was emptied into the hoppers, two laborers assigned to the duty gathered the sacks, shook them off to one side, packed them in bundles of 50, and stored them in the mixer cement house until from 20 000 to 30 000 sacks had accumulated, when a carload was shipped to the mills. The sweepings were sacked and used with the other cement.

Compressor Plant.—Practically all derricks, hoists, small pumps, air drills, small wood-boring tools, etc., were operated by air at a pressure of 85 lb., furnished by a central compressor plant which was near the mixing plant. The boilers of this plant also furnished steam for driving the mixing plant.

Two 24 by 30-in. straight-line air compressors, each with a capacity of 1 225 cu. ft. per min., furnished all the air. These machines were leased with other equipment from the contractor. The compressor and the mixer plant were supplied with steam by four 125-h.p. horizontal, locomotive-type boilers, leased from the contractor.

The compressors delivered air at 85 lb. pressure through a 6-in. main to a storage tank, 4 ft. 9 in. in diameter, 12 ft. long, and of

⅜-in. metal, located outside the building. A system of pipes was arranged to spray water constantly over this tank to keep it cool, and a special grade of oil was used in the compressors on account of the likelihood of explosion in the storage tank. The air was distributed to the principal points of use through a 6-in. main.

The construction of the compressor plant was commenced on November 16th, 1912, and the first compressor was turned over on January 14th, 1913. The plant was entirely completed and ready for business on February 1st, 1913.

Machine and Carpenter Shop.—The machine, carpenter, and blacksmith shops were all in one long building in the yards near the main storehouse. One line shaft, 84 ft. long, ran through the length of the building, and all machine tools were driven from it. The engine driving this line shaft was an 11 by 14-in., horizontal, single-cylinder engine owned by the Alabama Power Company. It was supplied with steam by a 30-h.p., vertical boiler, leased from the contractor. The engine and boiler-room was in the center and at the rear of the shops.

The machine shop contained the following tools:

One 30-in. radial drill press,
One 24-in. shaper,
One 18-in. by 10-ft. lathe,
One 1½-in. bolt threading machine,
One No. 94 Forbes, 2½ to 6-in., pipe cutting and threading machine,
One 16-in. by 5-ft. lathe,
One 30-in. diameter by 5-in. face power grindstone, and
One No. 4 Vulcan emery grinder.

The blacksmith shop contained the following:

One No. 2 Champion blower for forges,
Three blacksmith forges, home made,
Three anvils, from 250 to 300 lb.

The carpenter shop contained:

One No. 8 variety rip-saw and joiner,
One band saw,
One No. 402 bench grinder,

One No. 8 planer and matcher. This planer was very useful, and surfaced all form lumber; if the saw-mill had been operated by the Company, however, the planer should have been there.

The writers consider that the foregoing equipment served its purpose well, but there should have been added to the machine-shop a power hack-saw, a set of shears large enough to cut rounds up to 1 in. in diameter and flats up to 1 by 3 in., and a small steam-power hammer, on a job where there are a number of locomotives and cars to be taken care of.

The shops, as located, did not have sufficient floor and working space, and should have been separated from each other. Both should have been closer to the dam, and served by independent spurs from the railroad, so that cars of lumber could have been delivered to the platforms and yards direct. The laying-out floor of the carpenter shop should have been large enough to build one complete scroll case form on it, which would have taken a platform 75 ft. square. Also, there should have been a second laying-out floor, at least 50 by 100 ft., for ribs and other work of that kind.

The machine shop, if possible, should be as close to the work as convenient for all purposes, and should have at least two tracks for its own exclusive use: one for bringing machinery, etc., to be repaired at the shop; the other to serve as a general rip yard track. Where a railroad is operated and engines are taken care of, a pit for dropping wheels should be provided on one of these tracks.

Locomotive Cranes.—Two 10-ton locomotive cranes were leased from the contractor, and used in all features of the work. They did the work well, but the type used had only four wheels, which was hard on the track when traveling around. Two 1½-yd. clam-shell buckets were furnished with these cranes, and were used for unloading, storing, and reloading from storage all sand and gravel, and for coaling engines, etc. They were also used to assist in the disposal of excavation from the dam and tail-race, handling loaded skips. They were also used in unloading and erecting the contractor's plant and in reloading it when the work was completed. They were found exceedingly useful all through the work, but were not quite big enough for all purposes. After the work was started a larger crane was purchased, having a 50-ft., fixed boom and a 30-ft., trussed extension. This crane had a capacity of from 20 to 30 tons at 12 ft. radius, and

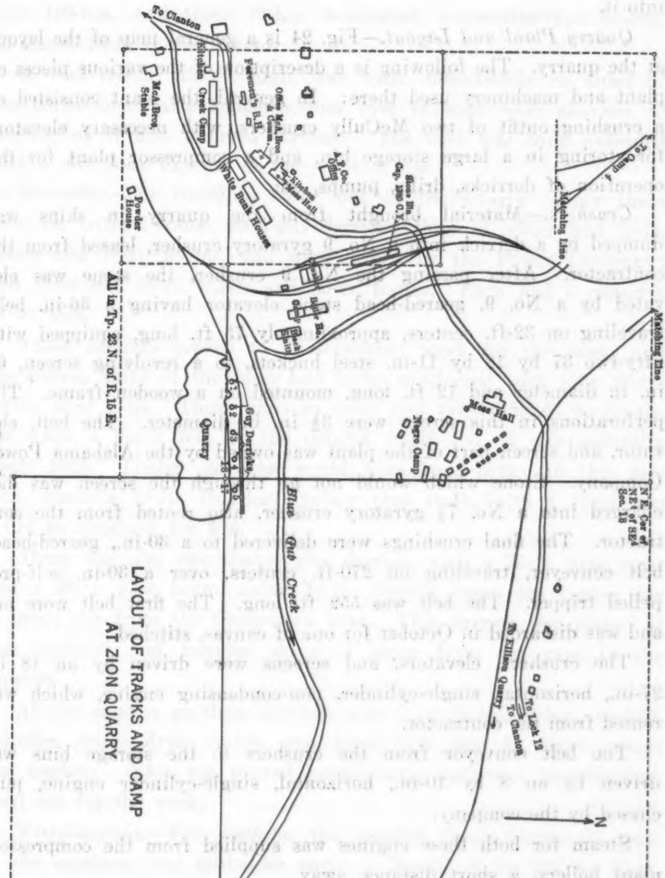


FIG. 24. LAYOUT OF TRACKS AND CAMP AT ZION QUARRY

was used at first for stripping at the quarry and loading cyclopean stone. Later, it was used in erecting the steel framework of the power-house and handling the heavy castings and machinery going into it.

Quarry Plant and Layout.—Fig. 24 is a general map of the layout at the quarry. The following is a description of the various pieces of plant and machinery used there: In general, the plant consisted of a crushing outfit of two McCully crushers, with necessary elevators for storing in a large storage bin, and a compressor plant for the operation of derricks, drills, pumps, etc.

Crushers.—Material brought from the quarry in skips was dumped by a derrick into a No. 9 gyratory crusher, leased from the contractor. After passing the No. 9 crusher, the stone was elevated by a No. 9, geared-head stone elevator having a 36-in. belt, traveling on 32-ft. centers, approximately 75 ft. long, equipped with fifty-two 37 by 18 by 11-in. steel buckets, to a revolving screen, 60 in. in diameter and 12 ft. long, mounted on a wooden frame. The perforations in this screen were 3½ in. in diameter. The belt, elevator, and screen part of the plant was owned by the Alabama Power Company. Stone which would not go through the screen was discharged into a No. 7½ gyratory crusher, also rented from the contractor. The final crushings were delivered to a 30-in., geared-head, belt conveyor, traveling on 270-ft. centers, over a 30-in. self-propelled tripper. The belt was 552 ft. long. The first belt wore out and was discarded in October for one of canvas, stitched.

The crushers, elevators, and screens were driven by an 18 by 27-in., horizontal, single-cylinder, non-condensing engine, which was rented from the contractor.

The belt conveyor from the crushers to the storage bins was driven by an 8 by 10-in., horizontal, single-cylinder engine, purchased by the company.

Steam for both these engines was supplied from the compressor-plant boilers, a short distance away.

Compressor Plant.—The compressor plant consisted of four 18 by 24-in., straight-line compressors, having a capacity of 670 cu. ft. per min., and were rented from the contractor. This plant could operate a maximum of nine drills and six derricks. The plant was

not quite large enough for the needs on this work, and one more boiler and compressor would have been more satisfactory.

The compressors and other engines were supplied with steam by four 100-h.p., and three 75-h.p. horizontal, locomotive-type boilers, all leased from the contractor.

A coal trestle, 12 ft. high, was built immediately in front of the boiler-room, and hopper-bottom cars of coal were shunted to the top of it and dumped through. The coal was thus unloaded with a minimum expense at the point of use.

Derricks.—For handling stone out of the quarry proper there were six latticed-frame, steel-guy derricks, three of which had 75-ft. masts and 70-ft. booms, and were owned by the Power Company; and three had 65-ft. masts and 60-ft. booms. Each had a capacity of 10 tons.

These derricks proved to be somewhat weak for the work, and the bottom third of the mast had to be reinforced with four 30-ft., 60-lb., steel rails. These derricks were also very difficult to repair, for when a boom fell, or the derrick itself fell, it was generally badly bent. Timber derricks would have been better in this section, where a 70-ft., long-leaf, yellow pine stock, with a 12-in. top could be obtained for $25.

There was also one timber guy derrick, with a 73-ft. pine mast and a 68-ft. pine boom, using 14-in. derrick irons, the timber for it being cut from the Company's land. This derrick was placed at the crushers, and handled skips of stone from flat cars to the crushers.

All the engines on these derricks were 7 by 12-in., 24-h.p., double-cylinder, double-drum hoists, and fitted with size 3½ Dake swinging engines. This rig proved satisfactory, and was considered a good one for the work.

Miscellaneous.—For washing the crusher muck before it went to the crushers, and cyclopean stone, a No. 11, 6 by 10½ by 18-in. pump, steam-driven, was set up in the boiler-room. It had a capacity of 450 gal. per min., and furnished water, at 60-lb. pressure, through three 2-in. hose lines with 1-in. nozzles. The men stood on a washing platform and directed the water on the skip loads of stone just before they reached the crushers.

For pumping the quarry pit an 8 by 5 by 13-in. pump, having a capacity of 100 gal. per min., was found amply sufficient.

The heavy drilling in the quarry was done with No. 14, gasoline, portable, blast-hole, well drills. These were driven by a single-cylinder, gasoline engine, and drilled a 5⅝-in. hole, the whole outfit being mounted on the same frame, on trucks.

The lighter drilling was done with Ingersoll-Rand, E 24, rock drills, equipped with No. 44 air heads, No. 42 chucks, and No. 27 tripods, with weights.

A blacksmith shop with two forges was maintained at the quarry for the running work there, and also a No. 3 Leyner drill sharpener, which took care of the sharpening of all drill steel. It was operated by air, and was a great money and time saver. Where a great deal of drill work is done, and there is a large quantity of steels to keep sharpened, a tool of this kind should be used.

As a whole, the quarry plant was satisfactory and the layout good. The plant had a capacity of 2 000 cu. yd. of crushed stone per day, but the largest day's run was a little more than 1 000 cu. yd.

Storage and Care of Materials.

Two yards were provided for the storage of sand and gravel, and were served by spurs leading off the main line of the railroad west of the yards at Lock 12, as shown on Fig. 23. Each spur branched into a double track after leaving the main line, so that a locomotive crane operating on one track could unload a string of cars on the other without having them switched. Structural steel was also unloaded and stored in one of these yards. In addition, 1 600 cu. yd. of sand were dumped through a high trestle next to the compressor plant and about 300 ft. from the mixer bins. Run-of-pit gravel (containing about 55% sand) from Elmore was stored in the same piles with the sand. Washed gravel was stored in separate piles. At the quarry a storage of 8 000 cu. yd. of crushed stone was accumulated during the latter part of the job, when the concreting slacked up at the dam. This stone was stored beside the main line of the quarry branch, being dumped from 7-yd. cars over the side of a 10-ft. embankment. Storing this material at that place permitted the quarry to shut down, thus getting rid of a large overhead expense some time before the concreting at the

dam was finished. This material was afterward loaded by a locomotive crane operating a ½-yd. clam-shell bucket. A maximum quantity of 8 400 cu. yd. of sand was in storage on December 14th, 1913, and approximately 2 000 cu. yd. of Elmore gravel, and 2 800 cu. yd. of washed gravel. These quantities of materials were put into storage primarily in order to avoid hauling over the railroad any more material than was absolutely necessary after January 1st, 1914, the rainy season generally starting about then and giving a great deal of trouble from soft track, derailments, etc.

Cement Storage.—Cement was stored in two warehouses, the main one being in the Lock 12 yard, and having a capacity of 15 000 bbl. This building was absolutely damp-proof, and the cement stored there was well preserved. The other warehouse, accommodating about 2 000 bbl., was built at the mixers. This storage fluctuated considerably, as it was used simply to insure a supply of cement at the mixers at all times. Most of the cement used at the mixers was unloaded there directly from cars and was not kept in storage.

Sand.—So much loam and foreign matter was contained in certain sections of the sand pit that it became necessary to place an inspector there. He sampled the sand from fixed positions in the cars and made the ordinary rough test of mixing it with water in large test tubes and allowing the material to settle after being thoroughly shaken up. The coarse sand settling to the bottom was measured, and the percentage of clay or loam resting on its surface was determined. Cars containing more than an average of 10% of clay or loam, as determined by this method, were not accepted. This percentage was greater than is ordinarily allowed, but there were times when no cleaner sand could be obtained, and many carloads were rejected. The concrete obtained with the sand was very dense and of excellent quality. A very close watch was also kept on foreign crushed stone, as some of it was dirty; if it was bad, it was rejected, by an inspector at Ocampo, before being brought to the dam.

Watch was also kept and inspection made of incoming materials at the dam. The engineers in charge on the dam made regular trips during their respective shifts to the bins and the mixers, and kept close watch on the character of the materials running.

A good deal of trouble was caused by using washed sand from foreign pits by itself; being very coarse, it would not hold the cement

or water, and would not dump out of the buckets, thus causing considerable delay at times. The principal remedy was to dump cars of fine sand containing some loam alternately with the coarse sand, mixing it as much as possible in this manner. About 30% of this coarse sand was retained by a $\frac{1}{8}$-in. screen.

A series of tests was run on the Elmore gravel, and also on the washed gravel, to determine its fineness; 50% of the washed gravel would pass a $\frac{1}{2}$-in. screen, and 55% of the Elmore run-of-pit gravel would pass a $\frac{1}{8}$-in. screen.

The quantities of the various materials were determined both by measurement and by weight. All materials coming through Ocampo were weighed on a standard track scale installed by the Power Company. One hundred cars of material of each class were weighed and also measured, and the weight per cubic yard was thus determined. This factor was then used on all subsequent receipts. All quarry-crushed stone was measured in cars at the quarry by the inspector. All cyclopean stone was measured on cars by the yard clerk at Lock 12.

Section C—The Prosecution of the Work.

Railroad.—Owing to the site of the dam being 12 miles from the nearest railroad, one of the first problems to be solved was that of building a construction railroad. Time was very limited in which to construct this railroad to the site of the dam, as it was August when it was decided to start the work at Lock 12, and a railroad, under the contract, was to be completed to the dam by November 20th, 1912. As a matter of interest, plant was delivered there on November 26th. This short time allowance was practically the governing feature in the selection of the route adopted.

An old abandoned lumber road, known as the Clear Creek Railroad, extended from Ocampo, a point on the Louisville and Nashville Railroad 39 miles from Birmingham (Fig. 3), toward the dam, a distance of 17 miles. By extending this road 5 miles, the site of the dam was reached, making a total length of 22 miles. Surveys and investigations indicated that this would be the best route by which to get a railroad to the dam in at least 60 days. It was necessary to rebuild about 45 trestles on the old road and to re-tie it. The rails were still on the original ties, never having been taken up.

An inspection of Fig. 3 at once brings up the question: Why was the line not brought down from Ellison Quarry along Yellow Leaf Creek to the dam site, thus reducing the haul from the quarries materially? The answer is that, at the time Zion and Ellison Quarries were selected, the main line had been extended nearly to the dam, and about 80% of the work on the extension was done. If the quarries had not been at Zion and Ellison, the natural route for the line would have been as built.

In August the Power Company started a force of men from Ocampo to rebuild the old line, and a large grading outfit started to grade at the end of this line toward the dam. The work was completed, and steel was laid to the dam by November 26th, when carloads of plant were moved in. The roadbed, however, was still in a very bad condition, and 3 months more were spent with heavy forces re-tieing, surfacing, and opening up the old drainage channels which had become clogged. A steam shovel was put in, and the last 12 miles of the line on the dam end was ballasted, with shale taken out by the shovel and with crushed stone from the quarry. The first 10 miles of the road ran through a very rough hilly country, and the line was exceedingly crooked and full of heavy grades, though the road-bed was good and needed no ballast. On this section the maximum curve was 35° on a 2½% grade, the maximum grade being 4 per cent. On account of these physical conditions, Shay engines were used entirely on this division, five in all being purchased or rented.

The operation of the railroad was quite a problem in itself. In the height of the season more than 250 men were employed on it alone, and on some days more than 100 trains of all classes were operated. The road was divided into two distinct divisions, the first 10 miles being the Shay Division, using the Shay engines only, and the last 12 miles, including the Quarry Branch, operated by rod engines. During the period of greatest activity, outside of switching service, five Shay engines and six rod engines were used. Operating out of Ocampo, a 40-ton Shay engine could haul only 2 cars of sand or 3 cars of cement to a train. On the rod-engine division, 5 and 6-car trains were hauled with 60-ton engines.

On the rod-engine division, 8 miles from the dam, a branch line ran down to the quarries, a distance of 2 or 3 miles, and the heaviest movement was between this junction and the dam. Railroad head-

quarters were established at this point. A regular camp, similar in character to all the others, was built, shops were put up to take care of the rod engines, and a commissary was maintained. The superintendent, master mechanic, car repairers, and all rod division trainmen, hostlers, shopmen, etc., lived there. All Shay division trainmen, machinists, etc., lived at Ocampo, where there was a smaller camp; a well-equipped shop had to be maintained at this point, also, to take care of the Shay engines.

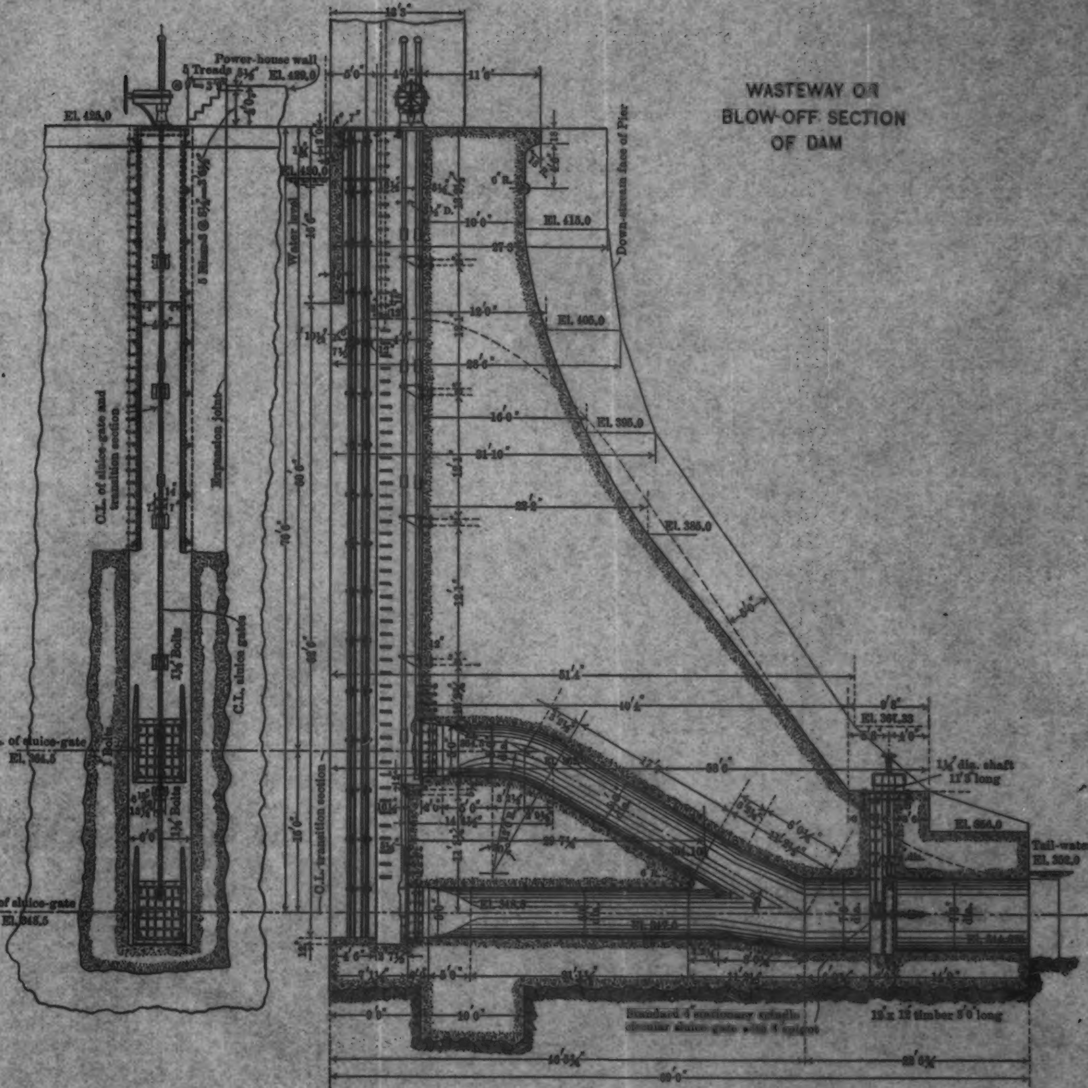
The Shay engines gave a great deal of trouble, their maintenance costs were high, and this division of the railroad was the most troublesome to operate. The use of Shay engines here was compulsory, as no other type could work on the grades and curves of the first 10 miles of the line. In view of the experience gained here, their use should be avoided, if possible. If a new line is built, it should be laid out for the use of rod engines throughout, and for heavy traffic. In the original road at this place 58½-lb. steel rails were used, but were too light. The writers would recommend nothing lighter than 75-lb. steel where traffic is handled directly in the original cars from a connecting railroad, because, in such cases, net loads of 50 and 60 tons will have to be taken care of in modern cars of 100 000 lb. capacity. The tracks should be well ballasted and the roadbed should be standard in every respect.

The time element entered largely into the selection of the railroad as used, making it almost compulsory. Had there been more time, it is quite probable that an entirely new and independent line would have been built from some near point on the Louisville and Nashville Railroad to the dam. The building of such a line, however, would have delayed the work at the dam from 60 to 90 days, and it was not considered good policy to do this, with the short time available in which to comply with Government requirements as to the completion of the dam.

Stream Control.—The first essential feature of the work that had to be studied and carefully worked out in detail was that of the control of the river. All indications and records of the behavior of the Coosa for 15 years previous indicated that this in itself would be no small part of the problem, and that its solution would have a most important bearing on the progress of the work. All the Government records for a period of 15 years previous to 1913 indicated a

PLATE XXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1334.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.

WASTEWAY OR
BLOW-OFF SECTION
OF DAM



minimum flow of about 2 000 sec-ft. in the dry season and from 90 000 to 100 000 sec-ft. in the rainy season. Table 2 contains these records. It was necessary, therefore, to determine just how great a flood should be taken care of without flooding Coffer No. 1. It was not desired to build a coffer-dam to withstand the maximum flood, but, at the same time, it was considered necessary to build one of such height as to withstand an average high flood during high-water season. This was fixed at about 70 000 sec-ft., and calculations indicated that a dam, obstructing practically two-thirds of the stream, with its top at Elevation 360 would pass that quantity before flooding. The discharges, as determined by gauging the river below later, showed that with the river at this elevation above the dam, 85 000 sec-ft. were passing. It was anticipated that Coffer No. 1 would be flooded, and this happened three times during the high-water period. The first coffer was built as determined, and the dam was concreted. In the section of the dam concreted, however, three permanent culverts were left in the waste-way section, as shown on Plate XXX, to fulfill a Government requirement calling for the passage of 4 200 sec-ft., with the water at Elevation 406. In the spillway section of the dam, there were left eleven tunnels, each 15 ft. wide and 16 ft. high, capable of passing 20 000 sec-ft. at Elevation 358, which was the elevation of the top of Coffer No. 2. From the top of each of these tunnels, two shafts ran upward through the spillway section and were used for filling them when the gates in front of them were closed. When the work inside Coffer No. 1 was above Elevation 360, this coffer was cut out and the river was turned through the eleven 15-ft. openings. Coffer No. 2 was then completed, and the work in it was finished. The work in Coffer No. 2, it was anticipated, would be accomplished during the low-water summer season, and the eleven openings would be ample to take care of any quantity up to 15 000 sec-ft., without flooding the coffer. The charts of previous years showed one or two flashy floods of from 15 000 to 25 000 sec-ft. during this average period. There was one flood of this character before the work inside the coffer was finished, water coming over into Coffer No. 2, delaying a portion of the work therein about a week, in October, 1913.

In the section of the dam constructed inside Coffer No. 2, ten 8 by 12-ft. openings were left, their bottom elevations being 357,

which was approximately 4 ft. below the top of the large openings left in Section No. 1. These openings were left as a positive safeguard to the power-house end of the work. This portion of the work was much slower than that on the dam, and it was feared that it would hang over until January, which was a very doubtful season, and a month in which a great deal of water might be expected. It was feared that, with only the eleven 15-ft. openings, a heavy and protracted flood would back up the river and there might be a rise sufficiently high to flood out completely the power-house work. The ten 8 by 12-ft. openings would have prevented this and, at the same time, helped to draw down the river more rapidly, if it rose to a considerable height.

Two types of gates were used to close these openings, a detailed description of which is given elsewhere. Figs. 15 and 16 give an idea of how these openings were arranged and closed.

Coffer-Dams.—The construction of Coffer-Dam No. 1 was begun on September 11th, 1912. It was composed of cribs 20 ft. square, roughly divided into two compartments; an 8-ft. compartment on the outside for clay and loam, and a 12-ft. compartment on the inside for rock. The cribs were built on the shore, launched, towed to their places, and sunk. After getting about 300 ft. out from shore, it was found necessary, on account of the increasing current, to stretch a $\frac{3}{4}$-in. wire rope across the river, by which the cribs were trolleyed out and placed. Before building a crib, soundings were taken, and the bottom timbers of the crib to be placed at that point were framed to conform as closely as possible to the shape of the bottom of the river. The coffer-dam was built of separate cribs to about Elevation 353. The joints in the timbers from that elevation to the top were then broken in the center of the cribs so as to make it as strong and as near one unit as possible. Sheeting, consisting of two thicknesses of 2-in. plank with lapped joints, was then placed on the outside of the cribs, and a toe-fill of sand, gravel, and loam, with a slope of about 1:1, was placed around the complete dam on the outside. Rock for filling and weighting the cribs of Coffer No. 1 was secured from a slate quarry about 200 ft. up stream from the dam, on the west bank of the river. Owing to the extreme swiftness of the current, the east or river end of Coffer No. 1, running up and down stream,

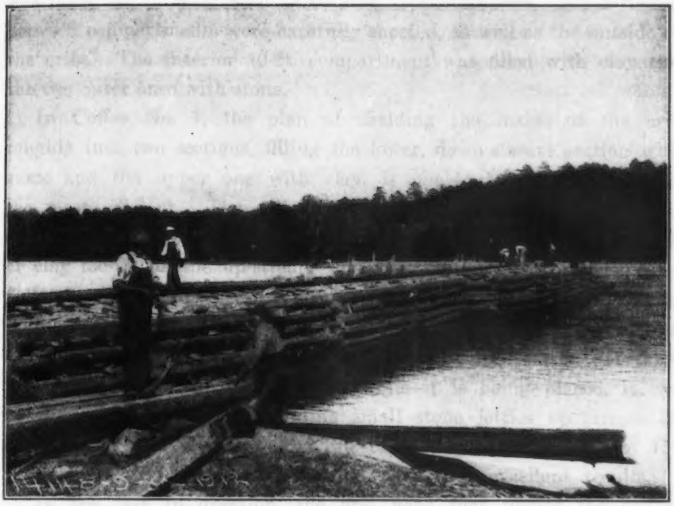


Fig. 25.—Coffer-Dam No. 1, Shortly after Inauguration of Work at Lock No. 12.

Fig. 26.—Coffer-Dam No. 1 Completed. Drilling of Foundation Started.

was made 30 ft. wide with three 10-ft. compartments. All divisions between compartments were carefully sheeted, as well as the outside of the cribs. The interior 10-ft. compartment was filled with clay and the two outer ones with stone.

In Coffer No. 1, the plan of dividing the inside of the crib roughly into two sections, filling the lower, down-stream section with stone and the upper one with clay, is not to be recommended as the most efficient. The cribs should be filled entirely with stone, and all stoppage of leakage should be made with a substantial earth or clay toe-fill on the up-stream side, care being taken to sheet carefully this up-stream face. When the toe-fill has been properly made, if there is much current, a 2 or 3-ft. layer of rough rip-rapping should be dumped on the outside to protect it from wash. If there is much wash of the earth toe-fill while it is being placed, it can be stopped effectively by building small stone jetties up stream 30 or 40 ft., at right angles to the line of the cribs, at intervals of 150 ft. or more. This scheme was used here with excellent results.

In the case in question, the first flood that topped the coffers took out a large portion of this inside fill, and the space formerly occupied by the earth had to be refilled with stone. It was also found that this interior fill was not effective as a leak stopper unless it was made like the outer river end, that is, in a tightly sheeted interior chamber which absolutely prevented any wash through it.

The first timber used in making the coffer-dam was squared on four sides, but, later, timber squared on two sides was used, which decreased the time of handling and expense at the saw-mills. All coffer-dam timber was hauled from the mills to the dam—about ½ mile—by teams. On Figs. 25 and 26 is shown the beginning of construction of this coffer and its completion. Fig. 27 shows typical sections of the various coffer-dams.

Pumps were started on December 3d, 1912, but numerous leaks developed under the down-stream coffer, and the pumps were shut down until the toe-fill could be fully placed along the south section. A portion of the river bed was finally unwatered by December 13th, 1912. Coffer No. 1 was entirely completed by December 21st, its total length being 1 640 ft. It contained 525 000 ft. b.m. of timber, and took a little more than 3 months to complete. The

filling for this coffer, as well as for all others, was carried in 1½-yd. cars, which were filled by hand, from quarries on each bank of the river, with stone and earth from earth pits in the bottom, and hauled in trains of 4 cars each, by a mule, out on the coffer as far as it was completed. This method proved satisfactory, and no great trouble was experienced.

On May 20th, 1913, a coffer-dam, 10 ft. wide, was started below Coffer No. 1 to encircle the tail-race excavation (see Figs. 23 and 27). As the water was very low at this time, it was not necessary

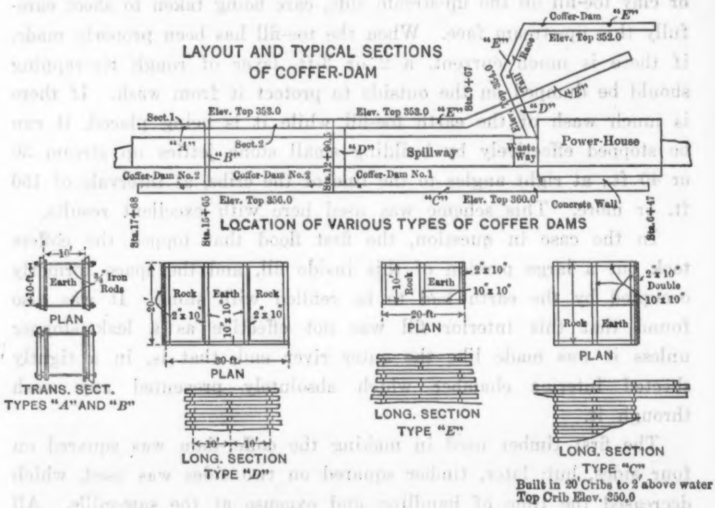


FIG. 27.

to build cribs on the shore, and they were constructed in place, the water being about 4 ft. deep. The average height of this coffer was 6 ft., the elevation of the top being 350.5 and its length 655 ft.

Coffer No. 2 was begun on June 17th, 1913, and the up-stream side was built of cribs of the same size and in the same manner as Coffer No. 1. A false deck was built above the water on this coffer and loaded with stone, allowing the water to run through the cribs until the up-stream section was practically completed and connection made with Coffer No. 1, when it was filled entirely with stone. This was done to keep the current as slow as possible to

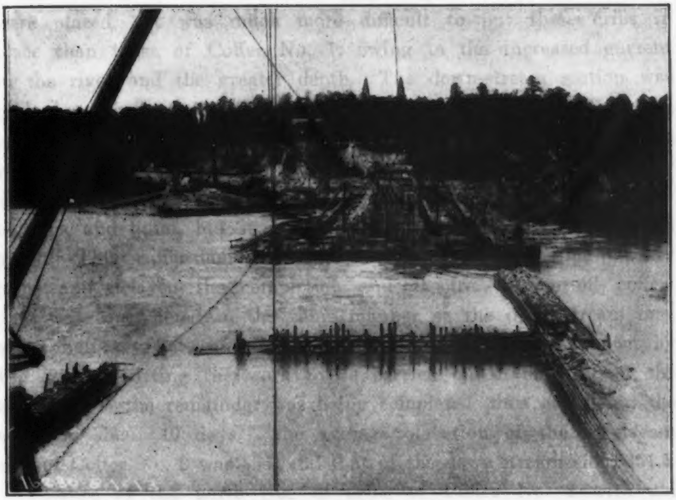


FIG. 28.—SECOND COFFER-DAM, SHOWING CONSTRUCTION OF LIGHT PARTITION RIBS UNDER WAY.

FIG. 29.—CUT-OFF TRENCH PARTLY FILLED WITH CONCRETE, SHOWING TYPE OF ROCK FOUNDATIONS.

avoid backing up the water and deepening it before all the cribs were placed. It was much more difficult to put these cribs in place than those of Coffer No. 1, owing to the increased current in the river and the greater depth. The down-stream section was built by using two 10 by 10-in. uprights with two 10 by 10-in. wales on the inside and 2-in. sheeting nailed on the inside of the wales, with only a loam fill. This makes a very cheap and quickly constructed coffer-dam, where there is not much head and the water is slack and quiet, but it is not very stable, and is easily washed away. This coffer-dam was partly constructed when high water came and delayed the completion and closure. A cut-off coffer-dam was constructed in the same manner as the down-stream one, about half-way between the shore and Coffer No. 1, as shown by Fig. 28, permitting the work to be carried on in one-half of the coffer, while the remainder was being completed, thus advancing the work by about 10 days. The average elevation of the up-stream side of Coffer No. 2 was 354, and that of the down-stream side, 351.5.

Closure of the north line of cribs of Coffer No. 2 with old Coffer No. 1 was made on August 20th, 1913. The closure in the south line was made about August 27th, 1913.

The total length of all the coffer-dams was 3 423 ft., and the total quantity of lumber and timber used was 898 984 ft. b.m.

Excavation in Dam and Power-House Foundations.—In general, the rock throughout all the foundation of the dam and power-house was a hard, blue, thin-bedded slate with numerous quartz seams running through it. Its outcrop crossed the axis of the dam at an angle of about 45° and dipped down stream, thus affording additional security from leakage and sliding. The general rough character of this bottom is shown by Fig. 29. Occasional soft streaks, from 6 to 8 ft. deep and from 25 to 75 ft. wide, were encountered, and these were worked entirely out, down to the hard blue slate. All rock excavation was drilled with Ingersoll, E 24 drills, operated by air. These drills made 2-in. holes, having a depth which varied according to the character of the rock. This feature was watched carefully by the engineers on the dam, and the depth of hole to be drilled in various sections was given to the drill foreman; the quantity of powder to be used was also specified. The quantity of dynamite varied from 1½ to 6 sticks of 40% Red Cross du Pont

dynamite per hole. In wet holes, Forcite gelatine was used. As a rule, the holes were placed on the corners of 3-ft. squares.

Only two bad mud seams of any size were encountered, one in each abutment. The excavation was carried back into the hill in each case until these seams pinched out and good rock showed. Some small seams were encountered, and these were cleaned out thoroughly and grouted under pressure. The cut-off trench was carried from 6 to 15 ft. below the general level of the excavated bottom; its excavation is described under the heading "Channeling". No differences in the character of the rock in the "cut-off" trench from that in the bottom, already described, were found, except that the rock was somewhat more compact in the bottom of the cut-off trench.

After the drilling, blasting, and excavation of the materials was completed, and the bottom was fairly clean and compact looking, a gang of 20 men was put in on a 75-ft. section and generally worked there 2 days, barring, wedging, and picking the surface of the work. This was continued until all loose slabs of slate were removed, and then scrubbing, washing, and cleaning were continued until the foundation was absolutely clean and free from all loose materials. Wire brooms were used all over the bottom, and a 2-in. fire hose, with a ½-in. nozzle, under considerable pressure, was used in washing and scrubbing. If, after cleaning the bottom, there still appeared to be pockets or layers of soft material, it was again drilled, shot, and excavated, and then finally cleaned up. Wherever leaks of any consequence occurred (which were generally low down in the "cut-off" trench), a cone of spalls and mortar was built, completely enclosing the leak, the water being led out of a 1½-in. pipe from the bottom of the cone and another pipe rising out of its top. When the cone had set, the drain from the side was plugged and the water was allowed to rise in the vertical pipe. This pipe was brought up until some 15 or 20 ft. of concrete lay over the leak, and then it was grouted under pressure up to 50 lb. Where several bad leaks were encountered, a well was formed in a suitable place in the "cut-off" trench, and all the surrounding leaks were led to it through 1½ and 2-in. pipes. A pump was set up nearby to keep the water lowered in this well until the concrete was well set. The well was brought up to such height as was necessary to bring the water level to a stand, then it was pumped

FIG. 30.—DEEP GRAVEL POCKET ENCOUNTERED IN FOUNDATION, BETWEEN UNITS NOS. 1 AND 2 OF POWER-HOUSE.

FIG. 31.—GENERAL VIEW, SHOWING METHOD OF HANDLING WORK AND TYPE OF WALL FORMS USED.

down, the forms and dirt were cleaned out of it, and grout pipes were brought up from the drains leading into the wall, and then filled with concrete. After the concrete had set, the leaks were grouted back, under pressure, through the original drain pipes. There were no serious leaks in any portion of the work.

The rock under the power-house was slightly heavier and more dense than in other portions of the foundations, but at the west end, between Units 1 and 2, and partly under Unit 1, a large cavity filled with sand, gravel, and slate boulders was found. This cavity extended the full width of the power-house up and down stream, and was about 50 ft. wide on top, narrowing to about 15 ft. at the bottom. It was wedge-shaped, with the small end down stream. Fig. 30 is a view of this hole. The lowest part of the hole was at Elevation 291.0, or 35 ft. below the bottom of the draft-tubes. The hole itself, the writers believe, was of the nature of an old "geological mill", and was worn out by the whirling around of gravel and sand in pot-holes, these pot-holes wearing eventually into each other. The hole was cleaned to the bottom, as a dentist cleans a cavity in a tooth, and then filled with concrete. The hole extended both up and down stream beyond the limits of the excavation, and may have been of considerable length. Two derricks were used in excavating it, handling 1½-yd. cars from the hole to a narrow-gauge track, the material excavated being used for a toe-fill on the outside of the coffer-dam. After solid rock was reached at the bottom, 16-ft. holes were drilled in various places to make sure that there were no caverns below, but the rock drilled solid and clean. The 2 200 cu. yd. of concrete necessary to fill this hole to the general level of the surrounding bottom was poured through a chute, which was built directly from the mixer.

The following quantities were excavated:

Dam............	Earth	3 379 cu. yd.
	Rock	15 442 " "
Power-house......	Earth	8 572 " "
	Rock	11 818 " "
Tail-race.........	Earth	8 029 " "
	Rock	11 245 " "
Total		54 845 cu. yd.

Excavation was started on the dam and power-house foundations on December 17th, 1912, and completed on October 20th, 1913.

Channeling the Cut-Off Trench.—A cut-off trench, 10 ft. wide, from 6 to 15 ft. below the general bottom of the dam, extending under both the dam and power-house, and well keyed into both banks of the river, was cut. Two channelers were used for both sides of a large portion of this trench.

Time was lost at the beginning of the work in starting these channelers, and it was seen that they would have to be supplemented in order that the excavation of the trench might keep up with that of the general bottom, and be ahead of the concreting. Two quarry bars were purchased and two Ingersoll, E 24 drills were set up on each, channeling in the power-house foundations. These drills put down holes 6 in. apart on the line of the trench. This method was not satisfactory, as the rock was badly shattered in blasting and the trench had to be carried much deeper than was necessary in order to get below the shattered rock. The channelers gave a clean cut and much better job all around, and are far superior to the drill work.

In the abutments only the up-stream face of the abutment was channeled by drills, and no actual cut-off trench was made, the whole excavation into the hill being practically such a trench. The thickness of the channeler steel varied from 1¼ to 2½ in.

Throughout the work on the cut-off trench, 40% Red Cross dynamite was used, with the exception of a few wet holes where 40% Forcite was used.

Disposal of Waste Materials.—At the beginning of the work, before the derricks were in position to handle the excavation, all excavated material was moved by cableways, in 4-yd. steel skips, to 7-yd. dump cars hauled by an engine on the Low-Level Line, the material being used to widen the embankment for that line. This method, however, was very slow and unsatisfactory, it being hard to stop and dump a heavy skip over a 7-yd. car. Later, the skips were landed on flatcars, taken about 1 000 ft. up the Low-Level Line, on a steep hillside, where a large guy derrick was erected to pick them up and unload them. A large part of the draft-tube excavation in the power-house was made in this way. When the traveling derricks were put in service on the up-stream coffer-dam, a track was laid on the coffer-dam and a dinky locomotive, handling 7-yd. cars, took the excavation from

these derricks. It was lifted out of the bottom in 4-yd., steel skips and dumped on a slanting platform built for the purpose. The material was then dumped over the side of a 10-ft. bank into the river about 200 ft. up stream from the dam. Later, these derricks landed loaded skips on flat cars, handled by a dinky engine and a locomotive crane, on the dump already mentioned, picked them off and wasted them down the bank into the river. Material in the excavation, out of reach of the up-stream derricks, was picked up by those traveling on the down-stream coffer-dam and passed across to the up-stream derricks which, in turn, disposed of it as described.

The tail-race excavation was handled and disposed of in a different way: Two traveling derricks started at the east side of the excavation and backed off toward the west as they cleaned up the excavation. The bottom was drilled, shot, and then placed by hand in 4-yd. skips. These were picked up by the derricks and discharged into 7-yd. dump-cars, which were hauled out of the bottom by a cable operated by a regular hoisting engine, and disposed of down stream about 500 ft. from the side, on an embankment. All the tail-race excavation was made in this manner very quickly and satisfactorily.

In general, the following criticism is offered of the methods used on this work: A cableway is not satisfactory in handling the excavated materials, as it is not only expensive, but slow. A better scheme for the work in Coffer-Dam No. 1 would have been a system of standard-gauge tracks laid down from the west bank of the river, through the tail-race, to its extreme east end. The excavation should then have started at that end, backing off toward the west, the spoil, in skips, being lifted with the derricks and discharged into 7-yd. dump-cars, hauled by dinky engines. The power-house excavation could have progressed rapidly, also, at the same time, and both cableways would have been free to erect derricks, get plant and timber out to the job, and handle concrete, thereby advancing the work greatly. The writers are of the opinion that, as a rule, the cableway should be restricted to moving plant, derricks, forms, and general work, and also such concreting as is advantageous to place with it, and that it should not be made the main method for the transportation of all materials on the job, as it was here.

Forms Used for the Dam.—The forms used on the vertical faces of the dam were of the cantilever type, built in sections, 12 ft. long

and 6 ft. high, with upright posts extending 6 ft. below the bottom of the sheeting. Figs. 29 and 31 give a general idea of these forms. All the sheeting was sized and dressed to $1\frac{3}{4}$ in. in thickness, and all vertical posts were made of two 4 by 8-in. timbers nailed together, with 1-in. blocks between them at the top, middle, and bottom. The waling pieces were 4 by 8-in., or 6 by 8-in., laid flat. These forms were built complete and raised in one piece, from one lift to another, by a 3-ton chain block hung from an A-frame, guyed back with form wire to hook-bolts in the concrete. This particular kind of form was not used throughout the job, for, as the work progressed and carpenters became rather scarce, sections of this type were abandoned, and 3 by 6-ft. panels were made in the carpenter shop and simply tacked to the posts. This type was practically the same as the first and was handled more quickly and easily than the forms of heavier section.

The following method was used to hold the forms in place: When a form was completed, $\frac{3}{4}$-in. bolts, 22 in. long, with an ogee washer on one end, were placed in them, 10 in. below the top, the bolts having a 10-in. hold in the concrete when it was poured around them. Also, when the pour in the form was completed, and while the concrete was still soft, $\frac{3}{4}$-in. bolts, 12 in. long, with a hook on one end, were set in the concrete opposite each post and about 6 ft. back from the face of the form. When a section form or panel form was raised, it was lifted until the bottom of the sheeting was above the bolts set 10 in. below the top of the previous pour of concrete. The posts were fitted on these bolts which held the form in place, passing through the slot between the two 4 by 8-in. posts. A waling piece at the top of each section was used to wire the form back to the hook-bolts placed in the concrete. Wedges were placed at the bottom of the post to line up the form. The panel forms were handled in the same manner, the loose posts being placed on the bolts previously set in the concrete, and the panels dropped into place behind them and tacked to the posts. As the speed of filling the forms increased, two walings were used instead of one. All expansion joint forms were held in this same way. For the curved section of the dam, on the down-stream face, ribs were cut to the proper radius in the form shop and brought down to the job where sheeting was nailed on them in place at the dam. The ribs for all curved forms were made of two 2 by 8-in. timbers nailed together, with a 1-in. spacing block between them, leaving a 1-in.

FIG. 32.—FORMS FOR WASTEWAY OR BLOW-OFF CULVERTS.

FIG. 33.—SCROLL CASING FORMS IN PLACE DURING CONCRETING OF FIRST STAGE OF CONSTRUCTION.

slot to catch the anchor-bolts, in the same manner as the vertical forms. In some of the big blocks on the dam, where the concrete came up slowly on the forms, two 2 by 8-in. pieces were used in place of the two 4 by 8-in. timbers, and proved satisfactory.

As a general rule, these forms worked well and were fairly economical of timber. However, it is not the fastest method; after a form is poured, sufficient time has to be allowed for the concrete to set properly and for the bolts to take hold. More rapid work could be done with a form of the same general type, but built up continuously, releasing the bottom sections as the opportunity arose. In building the forms continuously, one could be raised as soon as the carpenters could walk on the concrete, and the next pour could be made from 12 to 24 hours sooner than with the type used. The latter, however, would take considerably more timber, but the speed would be increased, and in a country where timber is very cheap and speed a big consideration, the continuous type might pay. Fig. 32 shows the type of forms used in moulding the wasteway culverts in the section of the dam adjoining the power-house.

Forms Used for Power-House.—All straight and vertical wall forms in the power-house were of the same type, and were handled in exactly the same manner as the dam forms.

All special or curved forms, such as for draft-tubes, scroll casings, and penstocks, were made in the form shop according to detailed drawings furnished by the Company's engineers.

The type of scroll casing used (Fig. 10), was more or less of an innovation, and there was considerable discussion at the beginning of the work as to the practicability of moulding such a complicated opening. The form work, as can be readily seen from Figs. 34, 35, and 36, was highly complicated, and, even when carefully detailed, was difficult of execution in the field, for several reasons: First, the average carpenter foreman is out of his depth on a complicated drawing of this kind; secondly, the setting and lining up of a large form of this character is difficult of execution; thirdly, unless the laying out of the forms is done carefully, they are hard to get out. In the case at Lock 12, the engineers made all the calculations and furnished a sufficient number of detailed drawings to enable the work to be laid out and assembled in the form shop. A competent engineer assisted the carpenter foreman in reading these drawings properly, and laying

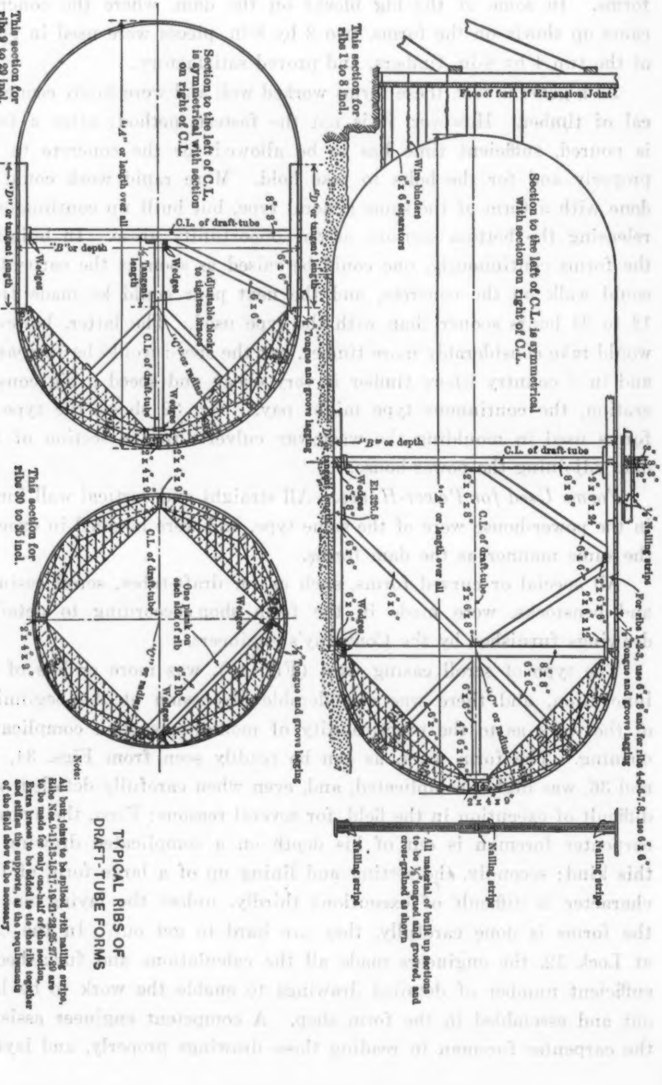

TYPICAL RIBS OF DRAFT-TUBE FORMS
This section for ribs 1 to 8 incl.
This section for ribs 9 to 29 incl.
This section for ribs 30 to 35 incl.
Section to the left of C.L. is symmetrical with section on right of C.L.
Face of form of Expansion Joint
C.L. of draft-tube
"B" or depth
Wedges
Nailing strip
Tongue and groove lagging
Note:

FIG. 34.

out the various parts of the form. The greatest care was exercised in setting up the forms and holding them in place. The work was handled in this manner with no great difficulty, and the final results were excellent.

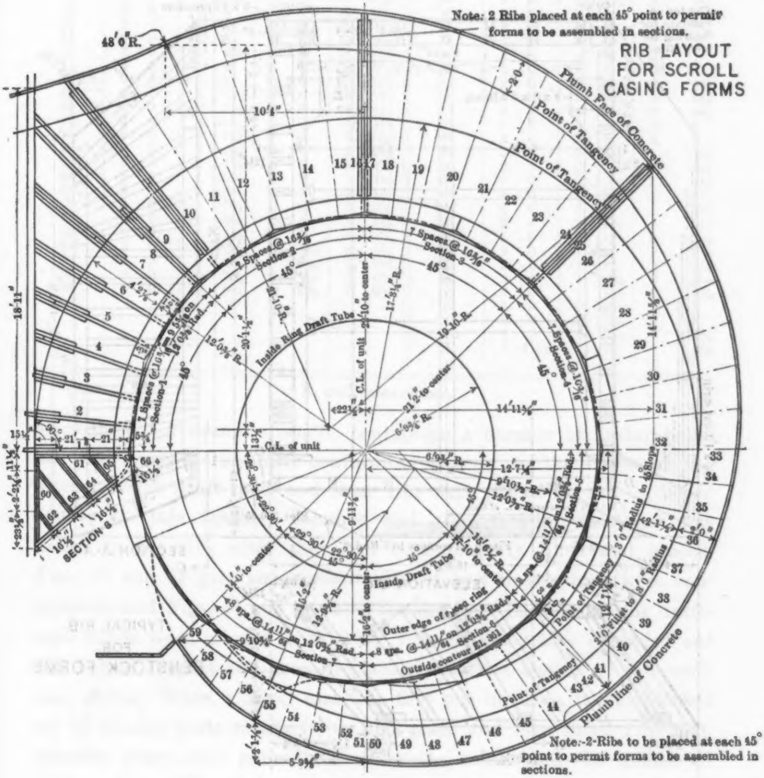


Fig. 35.

In detail, these complicated forms were built as follows: All the ribs of the scroll casings (Figs. 35 and 37), were made according to the drawings furnished, and were then set up in the shop in their proper positions, as laid out, and sheeted with 1-in. material. The

detailed dimensions for the ribs are given in Table 6. On the curved portions, 1 by 1-in. lagging was used, in order that it might bend easily and conform to its intended shape. The whole scroll casing was built in the shop in eight sections in order to facilitate handling

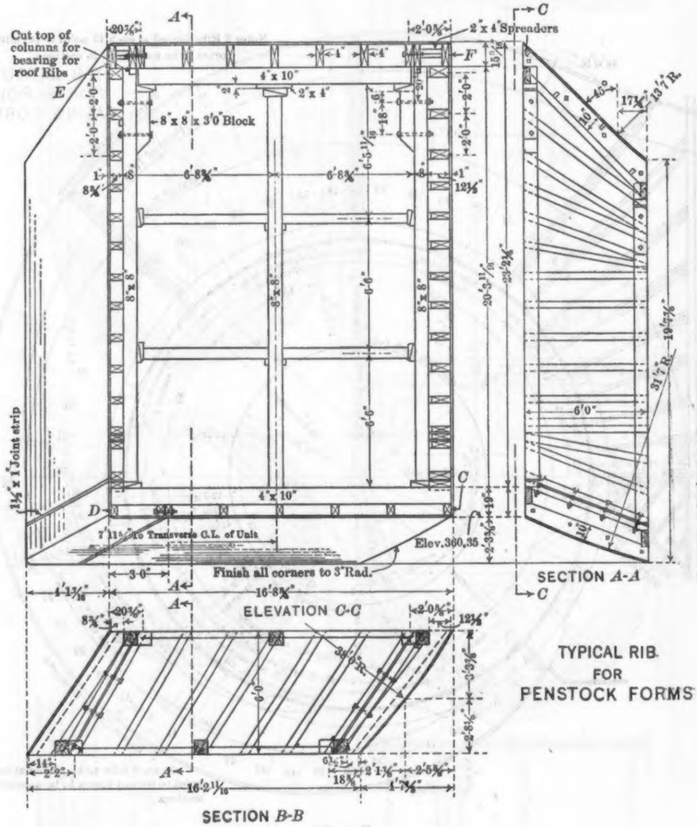


FIG. 36.

and erecting, each section being complete in itself. For the draft-tubes (Figs. 34 and 38), all the ribs were made and fitted together on the form-shop floor; they were then taken down and hauled to the dam and re-assembled on a platform from which a derrick could lift and

TABLE 6.—DETAILED DIMENSIONS FOR RIBS OF SCROLL CASING FORMS.

Rib. No.	A		B		C		D		Rib No.	a		b		c	
	Ft.	In.	Ft.	In.	Ft.	In.	Ft.	In.		Ft.	In.	Ft.	In.	Ft.	In.
10	2	2½	5	4¼	0	6	12	9⅝*	33	2	0	5	3⅝	4	0
11	2	1	4	7½	0	6	11	11⅜*	34	2	0	5	0¾	3	9½
12	2	0	4	4	0	6	11	6⅞	35	2	0	4	10	3	7½
13	2	0	4	2	0	6	11	4⅞	36	2	0	4	7¾	3	5¼
14	2	0	4	0	0	6	11	2⅞	37	2	0	4	5½	3	3¼
15	2	0	3	9½	0	6	11	0⅝	38	2	0	4	3	3	1
16	2	0	3	7	0	6	10	9⅞	39	2	0	4	1	2	11
17	2	0	3	7	0	6	10	9⅞	40	2	0	3	11¼	2	8¾
18	2	0	3	4½	0	6	10	7⅛	41	2	0	3	9⅝	2	6⅜
19	2	0	3	1¼	0	6	10	4⅛	42	2	0	3	9⅝	2	6⅜
20	2	0	2	10¼	0	6	10	1⅛	43	2	0	3	8	2	4
21	2	0	2	7¼	0	6	9	10⅛	44	2	0	3	6	2	2
22	2	0	2	4¼	0	6	9	7⅛	45	2	0	3	3½	1	11¾
23	2	0	2	1¼	0	6	9	4⅛	46	2	0	3	1¼	1	9¾
24	2	0	1	10½	0	6	9	1⅜	47	2	0	2	10½	1	7¼
25	2	0	1	10½	0	6	9	1⅜	48	2	0	2	8	1	4½
26	2	0	1	7¾	0	6	8	10⅝	49	2	0	2	5	1	1¾
27	2	0	1	4¾	0	6	8	7⅝	50	2	0	2	1⅝	0	10½
28	2	0	1	1½	0	6	8	4⅜	51	2	0	2	1⅝	0	10½
29	2	0	0	10¼	0	5⅛	8	1⅛	52	2	0	1	11	0	7½
30	2	0	0	6¾	0	3⅜	7	9⅝	53	2	0	1	8	0	4½
31	2	0	0	3¾	0	1⅞	7	6⅝	54	2	0	1	4¾	0	1½
32	2	0	0	0¾	0	0⅜	7	3⅝	55	2	0	1	1½	0	1¾
									56	2	0	0	10½	0	5
									57	2	0	0	7¼	0	8
									58	2	0	0	3½	0	11¾
									59	1	11½	0	0	1	3¾

* R. generates ellipse.

place them. A concrete cradle, containing a number of anchor-bolts, had previously been built, conforming exactly to the curve of the bottom of the draft-tube, and these ribs were placed on a 1-in. board resting on this cradle. Tongued and grooved 1-in. sheathing was nailed to the ribs after they had been properly lined and braced. Figs. 33 and 39 give an excellent idea of how these forms were constructed, and Figs. 34 and 38 show the details. The tops of the draft-tube forms were covered with 2 by 6-in. planks which extended out and helped support and keep in place the bottom section of the scroll case above. When a scroll case was first put in place, it was blocked up on wooden posts to exact line and grade and then small 1 by 3-ft. concrete piers were poured up under the joints and the middle of each section. These piers also contained anchor-bolts, and after the concrete of the piers had set, the forms were bolted tightly to them. They were also wired down to hook-bolts set in the concrete below, to keep the form from floating. Penstock forms, Fig. 36, were made in sections, 6 ft. long, in the carpenter shop; they were then knocked down and transported on cars to the dam, where they were re-assembled

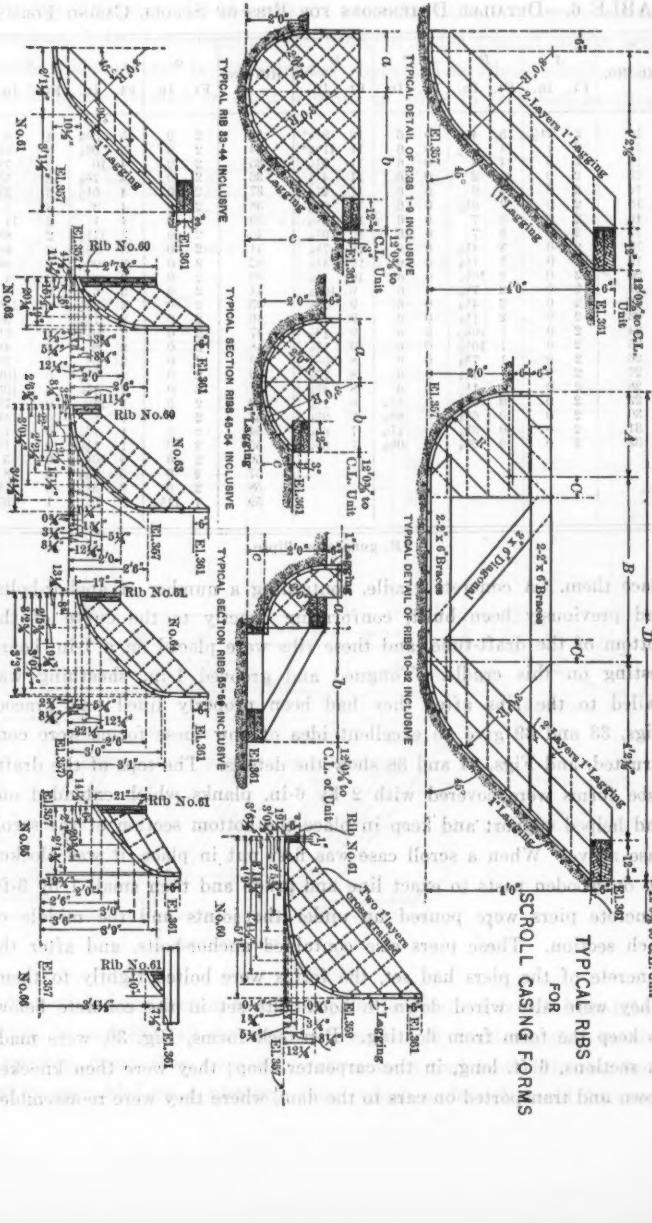
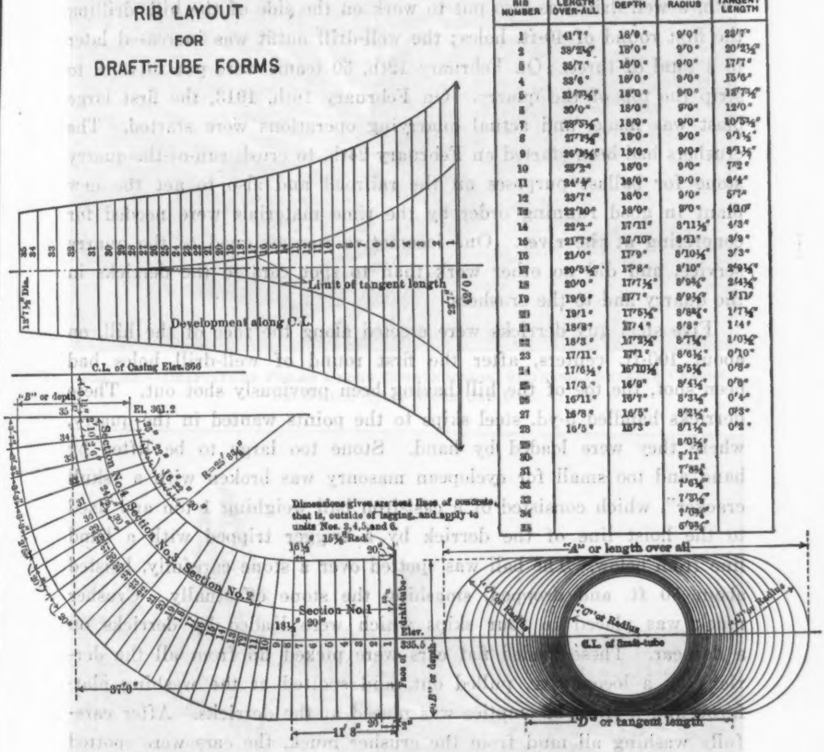

TYPICAL RIBS FOR SCROLL CASING FORMS
TYPICAL DETAIL OF RIBS 1-9 INCLUSIVE
TYPICAL DETAIL OF RIBS 10-32 INCLUSIVE
TYPICAL RIB 33-44 INCLUSIVE
TYPICAL SECTION RIBS 45-54 INCLUSIVE
TYPICAL SECTION RIBS 55-59 INCLUSIVE

FIG. 37.

on a platform adjoining a derrick which could pick them up and place them. A double cradle with anchor-bolts in it was prepared beforehand, in the same manner as the cradle for the draft-tubes. Types of drawings from which these forms were built are Figs. 34, 36, and 37.

The system of building complicated curved forms on a large laying-



	A	B	C	D
RIB NUMBER	LENGTH OVER-ALL	DEPTH	RADIUS	TANGENT LENGTH
1	41'7"	18'0"	9'0"	23'7"
2	38'2½"	18'0"	9'0"	20'2½"
3	35'7"	18'0"	9'0"	17'7"
4	33'6"	18'0"	9'0"	15'6"
5	31'7½"	18'0"	9'0"	13'7½"
6	30'0"	18'0"	9'0"	12'0"
7	28'5½"	18'0"	9'0"	10'5½"
8	27'1½"	18'0"	9'0"	9'1½"
9	26'1½"	18'0"	9'0"	8'1½"
10	25'2"	18'0"	9'0"	7'2"
11	24'4"	18'0"	9'0"	6'4"
12	23'7"	18'0"	9'0"	5'7"
13	22'10"	18'0"	9'0"	4'10"
14	22'2"	17'11"	8'11½"	4'3"
15	21'7"	17'10"	8'11"	3'9"
16	21'0"	17'9"	8'10½"	3'3"
17	20'5½"	17'8"	8'10"	2'9½"
18	20'0"	17'7½"	8'9¾"	2'4½"
19	19'6"	17'7"	8'9½"	1'11"
20	19'1"	17'5½"	8'8¾"	1'7½"
21	18'8"	17'4"	8'8"	1'4"
22	18'3"	17'2½"	8'7½"	1'0½"
23	17'11"	17'1"	8'6½"	0'10"
24	17'6½"	16'10½"	8'5½"	0'8"
25	17'3"	16'9"	8'4½"	0'6"
26	16'11"	16'7"	8'3½"	0'4"
27	16'8"	16'5"	8'2½"	0'3"
28	16'5"	16'3"	8'1½"	0'2"
29			8'0½"	
30			7'11"	
31			7'8¾"	
32			7'6½"	
33			7'3½"	
34			7'0¾"	
35			6'9¾"	

FIG. 38.

out floor, and also all curved ribs for other forms at the carpenter shop, is recommended on similar work. The shop, however, as previously noted, should be much closer to the job than it was on this work. The concrete-cradle method of setting these large forms is recommended as the best and most accurate.

Quarrying Operations, Zion Quarry.—Fig. 24 shows the general layout of the quarry tracks and plant, and the general relation between the various portions of the work. The actual work of clearing and erecting the camp and plant at Zion Quarry was started on November 11th, 1912. On January 20th, 1913, a force was put to work stripping from the toe of the hill, and tripod drills were started. On January 20th, a well drill was also put to work on the side of the hill, drilling the first round of 40-ft. holes; the well-drill outfit was increased later to a total of three. On February 12th, 50 teams were put to work to strip the top of the quarry. On February 16th, 1913, the first large blast was made, and actual quarrying operations were started. The crushers had been started on February 24th, to crush run-of-the-quarry stone for ballast purposes on the railroad and also to get the new plant in good running order by the time materials were needed for concreting at the river. One locomotive was assigned to the quarry service, and did no other work than to spot cars to the derricks in the quarry and to the crushers.

Five steel guy derricks were erected along the face of the hill, on about 100-ft. centers, after the first round of well-drill holes had been shot, the toe of the hill having been previously shot out. These derricks handled 5-yd. steel skips to the points wanted in the quarry, where they were loaded by hand. Stone too large to be lifted by hand and too small for cyclopean masonry was broken with a "skull cracker", which consisted of a cast-iron ball weighing 1 ton and fixed to the hoist line of the derrick by a trigger tripped with a hand line from below. The ball was spotted over a stone carefully, hoisted about 50 ft. and dropped, smashing the stone effectually. Crusher muck was placed in four skips which were loaded by derricks on a flat car. These loaded flat cars were picked up from all the derricks by a locomotive, pulled out, and spotted at the washing platform, and a train of empties was placed at the derricks. After carefully washing all mud from the crusher muck, the cars were spotted to a large timber guy derrick which picked up the skips and dumped them directly into the No. 9 crusher. The dumping device was simply a heavy inclined platform, on which the skip was lowered and prevented from sliding down by a lug under the skip catching a lug on the back of the platform. The skip was held and the contents slid out. After passing through the crushers, the stone was

FIG. 39.—DRAFT-TUBE FORMS UNDER CONSTRUCTION, SHOWING CONCRETE CRADLE ON WHICH THEY WERE SET UP.

FIG. 40.—END OF HIGH-LEVEL LINE, SHOWING CABLEWAYS, CEMENT WAREHOUSE, AND MIXER BIN.

carried by a belt conveyor to a bin of 1 800-cu. yd. capacity, and drawn from this bin directly into hopper-bottom cars by gravity. (See Fig. 21.)

Well-drills were used for all the heavy work, being augmented in the bottom by tripod drills and Jap hammers. Well-drill holes varied in depth from 40 to 60 ft. These holes were drilled from 15 to 18 ft. back from the face and at about 18 to 20-ft. centers, 4 by 8-in. sticks of Forcite gelatine, 40% dynamite, being used. The largest size of Forcite obtainable was 4 in. in diameter, and the holes originally drilled to 6 in. were reduced to 5⅜ in. in order to get the full benefit of the explosive of this size. Only three rounds were taken from the face of the quarry at Zion owing to the heavy over-burden farther in the hill. After they had been cleaned up, about 6 ft. of earth, overlying the rock in a meadow directly in front of the quarry, was stripped back a distance of about 75 ft., and the quarry was then carried vertically downward 60 ft. for a total width of 100 ft. and a length of 450 ft. The seepage from a nearby creek was small and easily taken care of by a No. 6A, 8 by 5 by 13-in., 100-gal. pump. The stone was of excellent quality, and seemed to grow denser with the depth. A well-drill test hole indicated the thickness of the stone below the general surface of the ground to be more than 75 ft.

All crushing operations were done in daylight; and, while getting out the crusher muck during the day, the cyclopean stone was sorted as uncovered and stored in a pile alongside the derricks. At night the cyclopean stone was loaded on flat cars, and the night shift also carried on mucking and stripping operations. The quarry afforded only a small quantity of cyclopean stone, principally on account of the limited space and the necessity for getting out enough crushed stone to keep work going at the dam. It was not possible to take the time to get out the cyclopean stone, because by doing that the crushing would fall behind. In other words, the quarry was such that a sufficient quantity of either crusher muck or cyclopean stone could have been gotten out separately, but not both at the same time. Difficulties and uncertainties in the transportation of coarse aggregates from the outside would not allow that source to be depended on entirely, and reliance had to be put principally on the company's crushers. For these reasons the percentage of cyclopean stone at the dam was much smaller that it would have

been under more favorable circumstances. In this case, the percentage had to be sacrificed to other requirements which were considered more important.

Ellison Quarry.—Ellison Quarry, about 1½ miles east from Zion Quarry, did not prove a success, as stated previously, owing to large clay seams which appeared throughout the rock after shooting. This quarry was worked, however, from March 17th, 1913, when stripping was started, until August 31st, 1913, the same methods of drilling and shooting being used as at Zion Quarry. A locomotive was assigned to its service, as at Zion, and two steel guy derricks were worked. The crusher muck was hauled to the crusher at Zion and unloaded there in the same manner as at Zion Quarry. The stripping at Ellison was done by teams and a steam shovel, there being a heavy overburden against the toe of the hill.

Storage.—Whenever a surplus of crushed stone accumulated in the bins at Zion, it was drawn out into 7-yd. dump cars and stored in a pile alongside a 10-ft. embankment. About 8 000 cu. yd. were stored and afterward picked up by a locomotive crane, operating a ½-yd. clam-shell bucket. By storing materials, the crushing and quarrying operations were carried on continuously, and a shutdown at the dam did not affect the quarries.

Zion Quarry was closed on December 24th, 1913, all the camps were removed, and the plant was shipped out.

Concrete.—As many different classes of material were used in the work, the coarse-aggregate bin, having a capacity of 800 cu. yd., was roughly divided into two sections; all crushed stone was dumped in one end of the bin and all washed gravel and Elmore gravel, consisting of 55% gravel and 45% sand, were dumped in the other end. The sand section of the bin was separated from the coarse-aggregate section by a partition. The crushed stone and gravel were drawn to the conveyor belt at the same time, in about equal proportions, and were elevated into the coarse-aggregate hoppers above the mixers. The sand was drawn by itself and ran into a separate bin, also above the mixers. There was a charging hopper just above the mixers, and all materials were drawn from the bins above into this hopper, where they were properly proportioned.

The mixing of the concrete was watched by an inspector on duty at all times at the mixer plant, and he was constantly in touch with

the chief inspector on the dam, who also with his assistants kept close watch on the concrete coming to the dam.

Three mixtures were used: 1:3:6 in the main body of the dam and the heavy sections of the power-house; 1:2½:5 in the cut-off trench and bottom 2 ft. of the foundation of the dam; and 1:2:4 around the scroll casings, penstocks, and all reinforcing in the power-house. The greater percentage of crushed stone came from Zion Quarry, the remainder being brought from outside quarries and shipped to the dam by rail. In addition to the crushed stone, 31 034 cu. yd. of gravel were used, principally around the complicated curved forms in the power-house and in filling the stream-control openings.

In general, as shown by the analyses of a great many different mixtures used in the work, there was about 10 to 15% too much fine aggregate used. The writers, in view of the results obtained, however, do not consider this a bad feature. There were no honey-combed walls in either the dam or the power-house, and only a few seepage spots appeared in any places except at the vertical expansion joints, the showing in this respect being excellent. When broken off in large chunks, the concrete was dense and had a uniform texture; and when the forms were removed, the outside appearance of all walls was good, no finishing being necessary except in spots where pieces had been broken out by bolts, etc.

Expansion Joints.—At first, expansion joints were placed 108 ft. apart, and after two of them had been started, the distance was cut down to 72 ft. The 108-ft. block proved a little too large to complete satisfactorily as the concrete at one end set before it did at the other. All joints were painted with a heavy coat of hot Barrett pitch. These joints were very effective, as far as could be noted at the end of one winter, and no cracks of any kind developed in the dam.

Handling and Placing the Concrete.—The concrete was dumped from the mixers into 2½-yd. buckets on small cars, or directly into chutes. The cars were hauled by mules to the cableways, two cars being used for each mixer. There were four tracks under the cableways, drawing closely to each other in pairs at the mixers. See Figs. 22 and 40. The buckets were picked up by the cableways and placed at the points needed. No. 1 mixer also dumped directly into a chute. Chutes were used wherever possible to deliver concrete to buckets traveling on small cars operated with a hoisting engine. A large

portion of the power-house concrete was passed through chutes, and the system when properly laid out is most effective. The chutes delivered the concrete to buckets on cars which were taken to the dam, *via* the Low-Level Line and the up-stream coffer, by a dinky locomotive. This method was as effective and rapid as any used on the job. A great deal of the concrete was dumped directly from the cableways, and this method was fairly rapid. However, the maximum 10-hour run with cableways was seldom more than 400 cu. yd. The writers believe that cars and derricks, where it is possible to use them, are more efficient and rapid than cableways. Where narrow forms are to be concreted, the cableway is exceedingly slow and should not be used if any other method can be found.

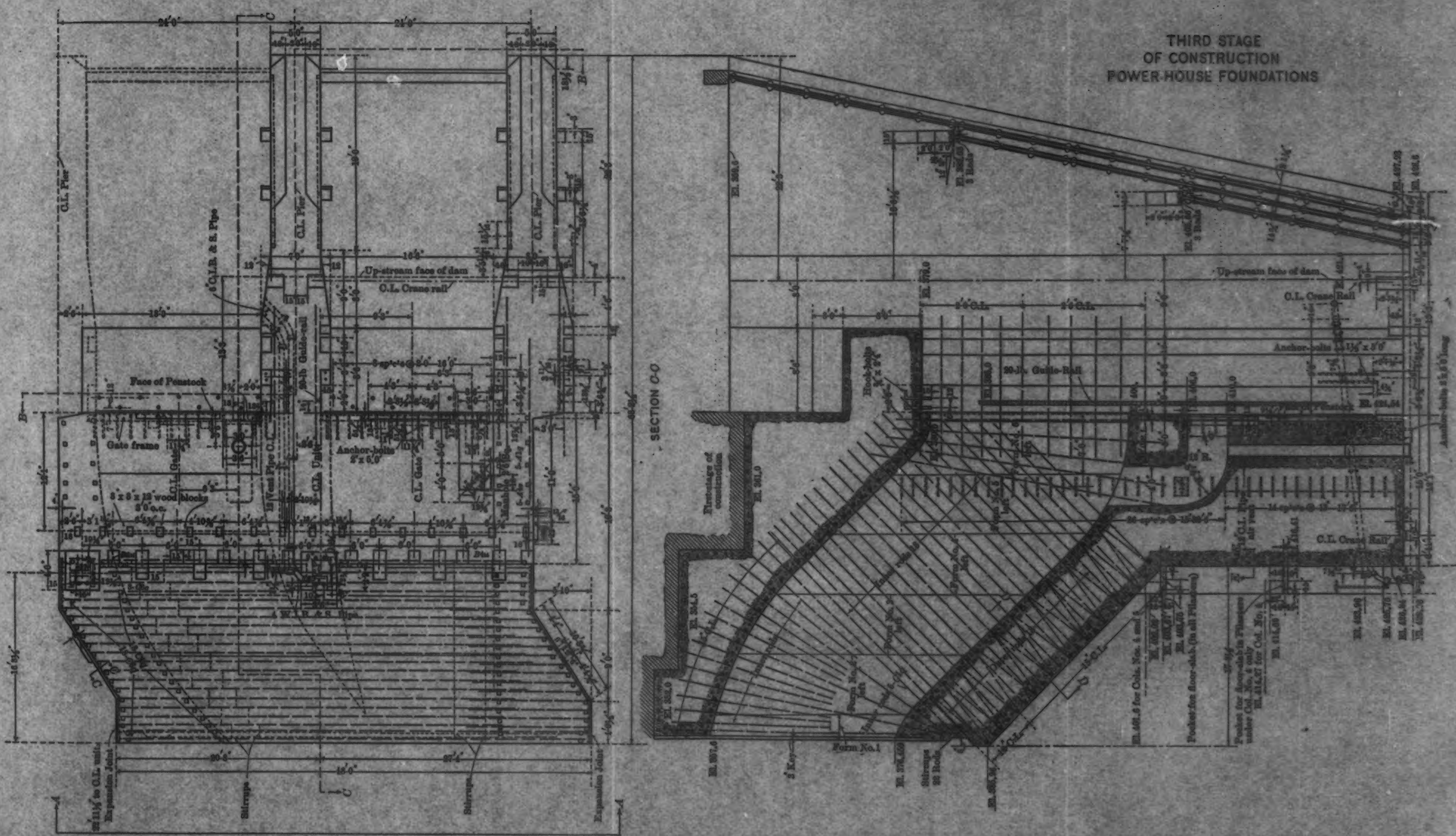
Cyclopean stones were delivered directly to the derricks on the coffer-dam, and, between buckets of concrete, were set by the derricks. Each stone was carefully washed and cleaned, and, after it was placed, it was carefully shaken with a bar and bedded. In the dam the percentage of cyclopean stone was small, for the reasons mentioned in discussing the quarry operations. The horizontal joints between pours were always left bedded with as many cyclopean stones as possible, in order to get a thorough bond with the next layer.

A total of 6 482 cu. yd. of cyclopean stone was placed in the dam, or 5.1 per cent. In the power-house foundations there were 1 188 cu. yd.

There were two types of chutes on the work. The first was made of No. 12 gauge metal and was 10 in. in diameter. It was of the semicircular type, and was served by a special bottom-dumping car at the mixers. The mixer dumped into the car, and the car was pushed by hand over a hopper to the chute, and dumped into it. This chute was not a success, the main objection being that it was too small. It clogged frequently, and took too many men strung out along it to operate it and keep it clear. The metal was too light and wore out too quickly. This size and type are not recommended for similar work. This chute was abandoned, and a home-made one was built, which gave satisfaction. It consisted of a trough, built in 6 to 8-ft. lengths, with sides about 18-in. high, and well braced; ⅛-in. sheet iron was rolled in 8-ft. lengths to a half circle, 14 in. in diameter, and fastened into the bottom of the trough, giving a chute with high sides and a rounded bottom. It was of sufficient size, discharged on slopes of 18° rapidly and without clogging, provided the concrete was wet

PLATE XXXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1394.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.

THIRD STAGE
OF CONSTRUCTION
POWER-HOUSE FOUNDATIONS



enough to flow, cost about two-thirds of the price of the first chute, and its life was nearly twice as long. It is to be noted that concrete discharged down a 75 to 100-ft. chute was as good as that which fell only a few feet from the mixer to the bucket, and on dumping the two side by side in the forms no difference could be discerned.

After leaving the mixer platform, the concrete was delivered to a derrick, *via* some one of the routes mentioned, and by it placed in the form. An inspector in the form saw that the concrete was brought up uniformly throughout, no racking being allowed. When it was seen that the form could not be completed in one pour, a temporary saw-tooth bulkhead was put in to prevent thin-edged layers. The inspector saw that all concrete was well worked after being dumped; when it lacked sand, it was worked with shovels and tamped until the mortar had worked its way all through it. All faces against forms were carefully spaded. Two bars were used in bedding the cyclopean stone in the soft concrete.

Before making a new pour on the surfaces of old concrete, they were thoroughly cleaned in the following manner: Two gangs of from 12 to 14 men with foremen, when the work was at its height, were kept cleaning forms only. Whenever possible, the surfaces of the old concrete were cleaned about 12 hours after pouring, while they were still green, all scum mortar and "laitance" remaining on top were carefully picked or shoveled off, and the surface was washed until it was absolutely bright and clean. Great stress was put on this particular feature. Concrete was not allowed to be placed in any form until it was cleaned to the satisfaction of the engineer. After the work got under way, and the amount of cleaning insisted on was fully comprehended by the contractor's superintendent, there was little trouble. Also, after a form had been properly washed and cleaned, a 1:2 mortar was dumped in and swept over the surface with a wire broom just before the concrete was dumped. Leaks through horizontal joints have not appeared in any portion of the dam or power-house, and the writers attribute this to the care exercised in cleaning and working the surfaces of old concrete as described.

Power-House Concrete.—In general, the power-house concrete was poured in the same manner as that for the dam, except that the mixtures varied, as noted elsewhere. Fig. 41 is a general view of the power-house operations on July 2d. The heavy blocks indicated in

the first stage of construction, Fig. 43, were of cyclopean masonry, the remainder being mass masonry. The power-house concreting was prosecuted in four separate stages, and complete details of each of these stages are given on Figs. 43 and 44, and Plates XXXI and XXXII, respectively. These drawings were followed closely in the field, and were of great assistance. The general plan to be followed was carefully discussed at the beginning of the work, and a line of action definitely decided. Fig. 42 shows the second stage of concreting completed. Fig. 45 shows the second stage completed and the forms for the third stage, that is, the concrete around the penstocks, being placed. In Fig. 46 these forms are shown erected. Fig. 47 shows the down-stream elevation of the power-house foundations, the photograph having been taken from the tail-race. Fig. 48 is a close view of the interior of the draft-tube; the man standing in the center of the opening giving a fair idea of its size at the exit.

General.—On April 12th, the first concrete was poured in Section 1 of the dam, immediately next to the wasteway section. Construction of the wasteway section followed immediately, and then the remainder of the dam in Coffer-Dam No. 1, and the heavy foundation blocks in the power-house, shown on Fig. 43 as Stage No. 1.

On May 13th, a night shift on concrete was organized at the dam. On May 26th, concreting in the last block of the dam inside Coffer No. 1 was started, and Fig. 31 shows the general condition at this time. On May 31st concreting was started in Section 6 of the power-house. On June 4th, No. 2 mixer was started and the concreting was begun on the bottom cradles for Draft-Tubes Nos. 5 and 6. On June 13th, the filling of the large natural hole in the foundations under Units 1 and 2 was started. By July 12th the concreting and other work had progressed to such a stage that the water in the river was turned through the tunnels left in Section 1 of the dam. On July 25th, Draft-Tube No. 1 was concreted to grade and ready for the scroll casing. On August 26th, concreting was begun in Coffer No. 2, next to the east abutment, which latter lagged behind on account of a bad mud seam which had to be worked out, and on which concreting was not started until September 22d. On September 5th, the first section of rollway in Coffer No. 1 was reached and poured, this marking the completion to Elevation 406 of a portion of the first section of the dam. On September 27th all the rollway section, inside Coffer

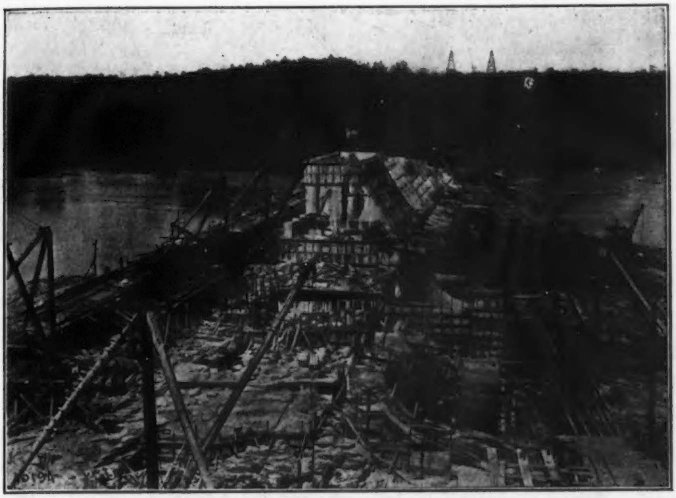


FIG. 41.—CONDITION OF WORK, JULY 2D, 1913. FIRST STAGE OF CONSTRUCTION OF POWER-HOUSE STARTED.

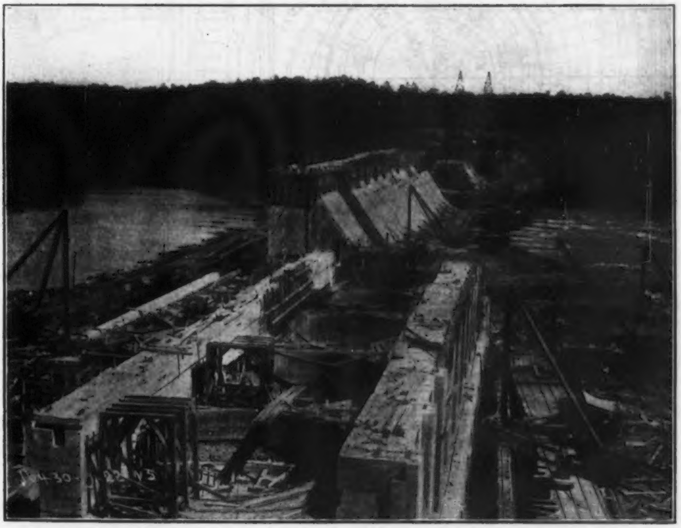


FIG. 42.—GENERAL VIEW OF WORK, SEPTEMBER 22ND, 1913. SECOND STAGE OF CONSTRUCTION PRACTICALLY COMPLETED; CONCRETING UNDER WAY.

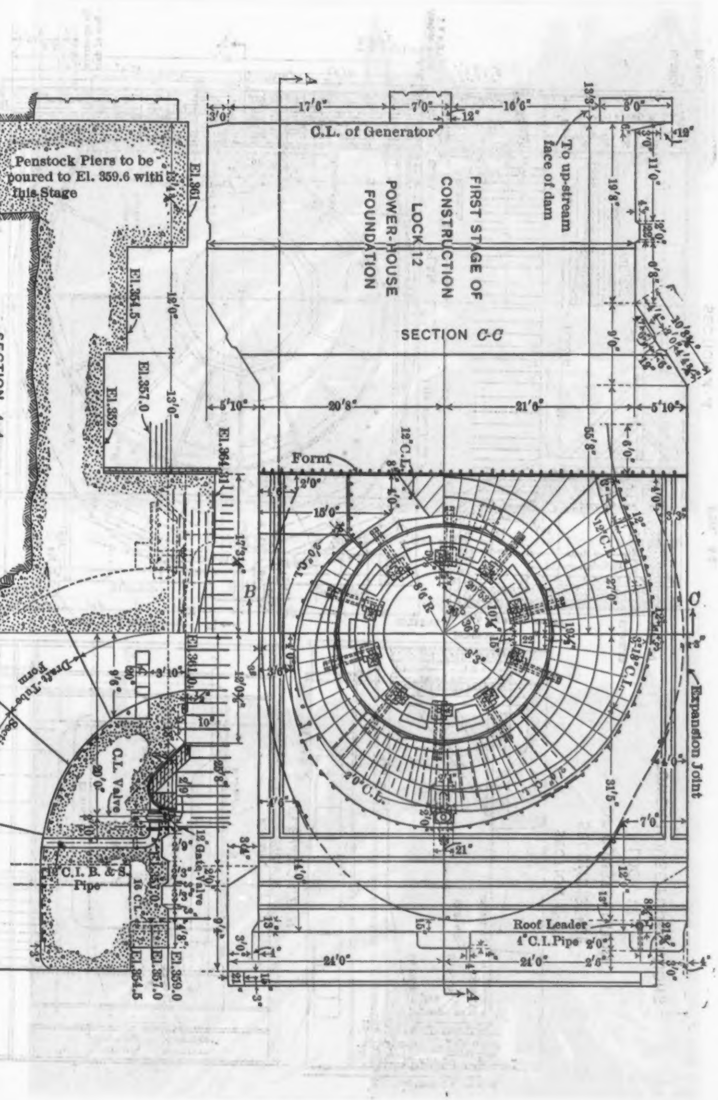

Penstock Piers to be poured to El. 359.6 with this Stage
El. 361
El. 354.5
El. 357.0
El. 352
El. 364.31
SECTION A-A
Draft Tube Form
Section
C.L. Valve
18" C.I. B. & S. Pipe
18" Gate Valve
El. 361.0
El. 354.5
El. 357.0
El. 359.0
C.L. of Generator
To up-stream face of dam
FIRST STAGE OF CONSTRUCTION
LOCK 12
POWER-HOUSE
FOUNDATION
SECTION C-C
Form
Expansion Joint
Roof Leader
4" C.I. Pipe

FIG. 43.

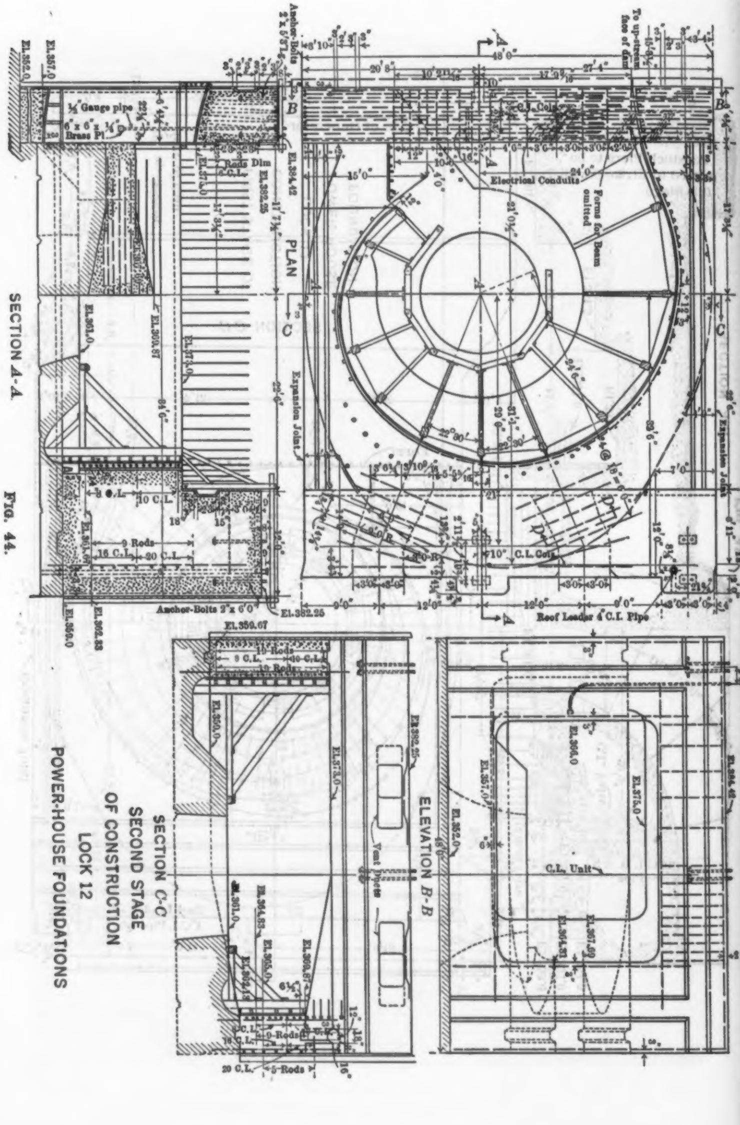


FIG. 44.

SECOND STAGE OF CONSTRUCTION LOCK 12 POWER-HOUSE FOUNDATIONS

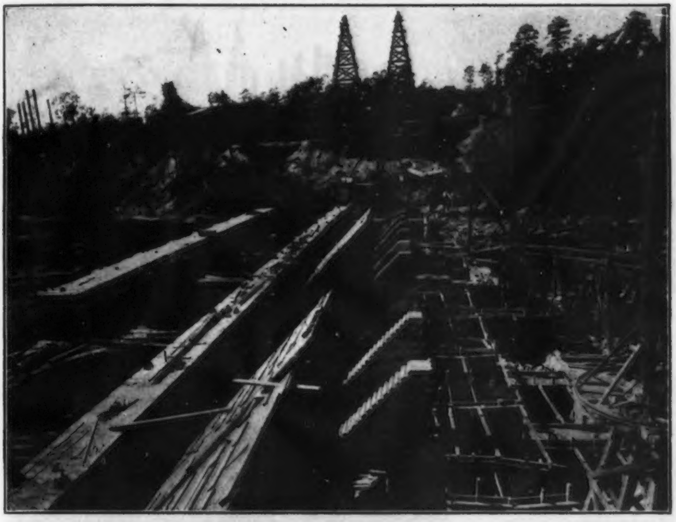


FIG. 45.—CRADLES FOR PENSTOCK FORMS. SECOND STAGE OF CONSTRUCTION COMPLETED.

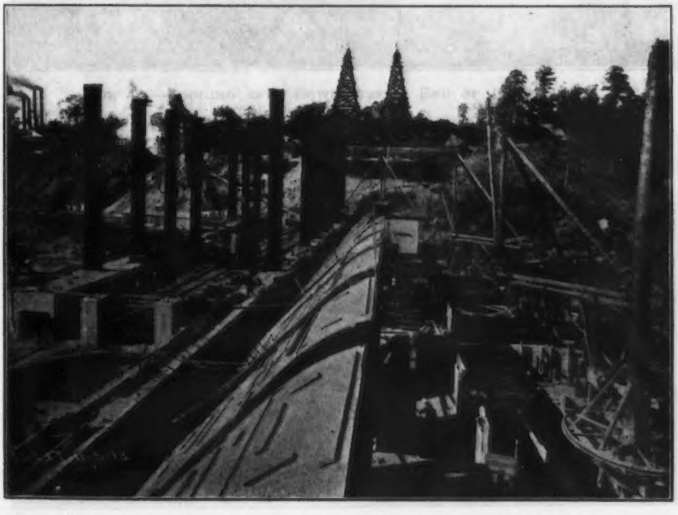


FIG. 46.—PENSTOCK FORMS UNDER CONSTRUCTION. LOCOMOTIVE CRANE SETTING UP COLUMNS OF POWER-HOUSE.

FIG. 47.—DOWN-STREAM VIEW OF POWER-HOUSE FOUNDATIONS, AFTER COMPLETION OF SECOND STAGE OF CONSTRUCTION.

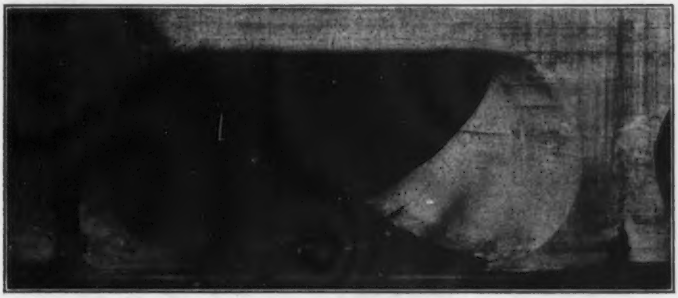
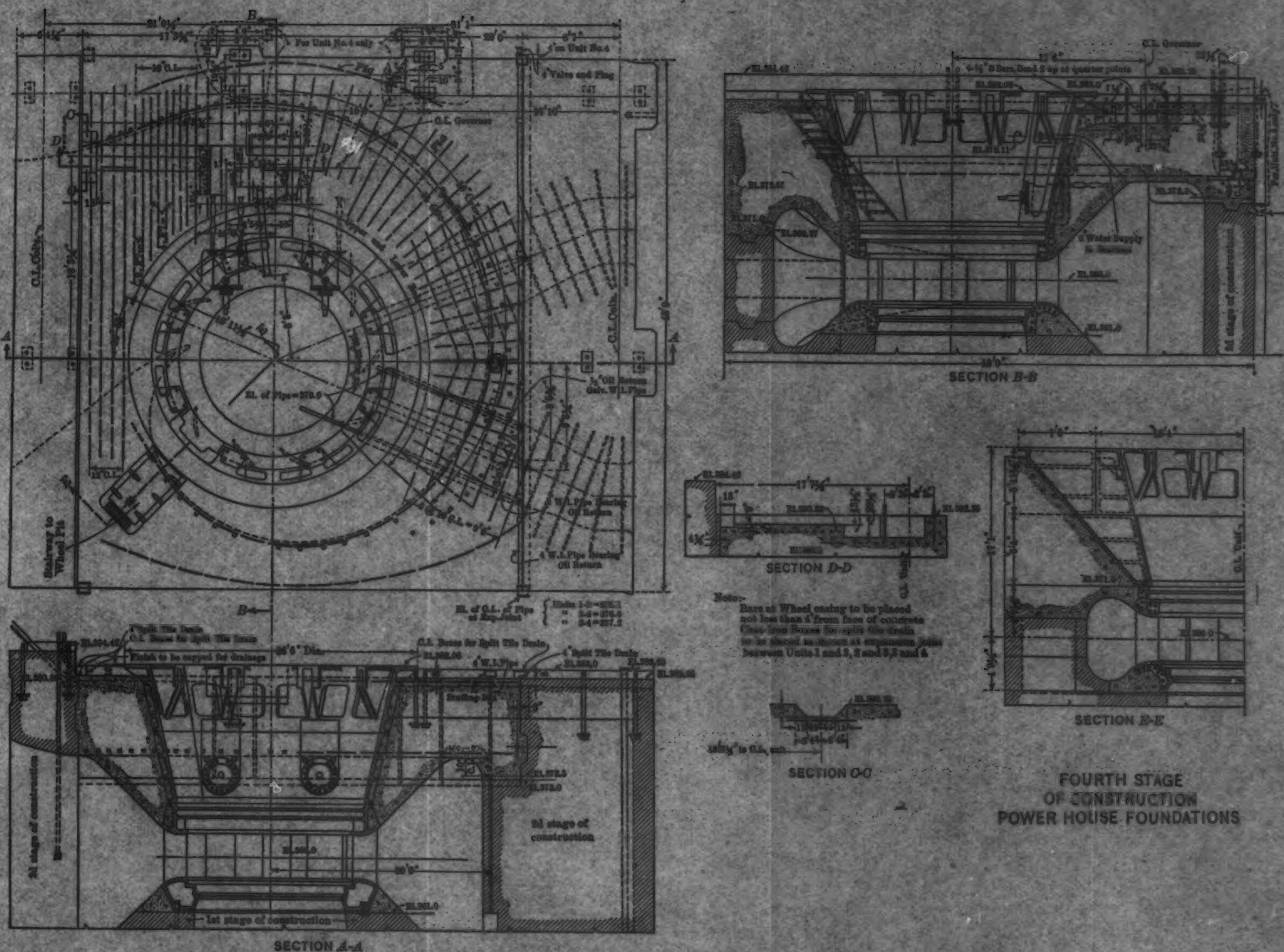


FIG. 48.—LOOKING INTO DOWN-STREAM END OF DRAFT-TUBE.

FIG. 49.—CREST OF SPILLWAY BEFORE CONSTRUCTING PIERS.

PLATE XXXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1334.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.



FOURTH STAGE
OF CONSTRUCTION
POWER HOUSE FOUNDATIONS

No. 1 (Fig. 49) reached Elevation 406.0, the crest, and the concrete all along the front of the power-house had reached Elevation 380. On October 13th, the highest run on the concrete was made, being 2 220 cu. yd. for two 10-hour shifts, the day shift alone putting in 1 120 cu. yd. in 10 hours. On October 18th, concreting was started around the first penstock forms, and on October 21st pouring was started on the last section of the dam (No. 9). On November 8th, the concreting of the bottom 4 ft. of the eleven 15-ft. stream-control openings was started. It was completed on November 21st. On November 25th, the dam for the entire length of the spillway was completed to Elevation 406, the crest. On December 20th the power-house concrete reached an elevation of 416 in front, and all the scroll casings, draft-tubes, etc., were ready for the setting of the speed rings. On January 3d, the filling of the stream-control openings was begun, and the work was completed on January 31st. On February 15th, the last wheel casing was set and the last scroll casing poured, completing the operating floor of the power-house to rough grade. On February 28th, the last concrete in the spillway piers was poured, completing the work, and on March 6th, the work of tearing out the mixers was started. From the day that concreting started to the last yard poured, 10 months and 18 days elapsed, and 187 802 cu. yd. of masonry were placed. Fig. 49 shows the finished rollway section before the piers were constructed.

Spillway Piers.—The spillway piers above Elevation 425 were so small in plan and contained so much iron framework that it was very dangerous to dump concrete into them from the cableways. A small derrick was rigged up on a standard flat car—the hoist being operated by air, as usual—for the pouring of these small piers in three 6-ft. lifts each. The cableway delivered concrete to this derrick car. Some 18-in. I-beams, which were bought for the purpose of carrying the traveling gate-operating mechanism between the piers, were used temporarily to span the back of the main piers at Elevation 425, and carry a standard-gauge track for the derrick car. These piers are shown in plan and section on Fig. 19.

Discussion of Methods Used.—In general, as previously stated, the writers do not consider cableways as the most satisfactory method of handling concrete, in view of the experience gained on this job. The system of belt conveyors and elevators for concrete materials is one to be avoided, if possible and if space permits, as a fruitful source

of delay. There was one delay of a week on this work, due to a broken elevator belt, and it was expensive. A gravity plant should be built, if it is possible to lay it out. Handling concrete by buckets on flat cars, with dinky locomotives to the derricks, is preferable to the cableway.

The plant, as laid out and used, was effective and, in 1 month, more than 35 000 cu. yd. of concrete were handled with it, but the writers believe that a gravity plant would have been an improvement and more effective. The point which it is desired to illustrate is that a gravity plant should be carefully sought first, and that cableways should not be made the main method of concrete transportation.

Reinforcement in the Power-House.—Complete and detailed drawings were furnished for the correct cutting, placing, and setting of all reinforcement. All curved rods were laid out on a large platform near the tail-race below the power-house, and there bent with a bender to the curves shown on the drawings. All stirrups, etc., were also made there.

Spacing rods were used, to which the regular reinforcing rods were wired. Large nails were driven into the forms, and the rods were wired to them in order to keep them at the proper distance from the forms. In some cases suspended frames were used to hold the rods in exact position until they were concreted. In all cases particular care was taken to keep the rods the proper distance from the forms and properly spaced. All concrete poured around the reinforcement was mixed in the proportion of 1 : 2 : 4.

Completing the Work.

Final Closing.—When all concreting in the dam had been finished, except the stream-control openings, and all piers had been concreted to Elevation 417 on the spillway, and the concreting on the power-house and west abutment to Elevation 420, it was deemed safe to close the gates. Fig. 16 shows these gates hung and ready to close. Previous to the closing, when it was seen that the work was reasonably safe, the west eleven 15-ft. openings in Section 1 of the dam were blocked off by placing wooden gates about 7 ft. high in front of them, one at a time, and a concrete wall, 4 ft. high and 6 ft. thick, was placed next to the bulkhead. This small wall was built first to control the leakage that came through these 7-ft. bulkheads, the water coming

through being collected in an interior box sump in the concrete and led through the wall down stream in pipes which were afterward grouted under pressure. The main part of the opening was then filled to the same height throughout as this front wall, no trouble from leakage being experienced. Each of these eleven openings was treated in the manner described, except that on the five westerly openings, which were built primarily for a stop-log gate, the permanent stop-logs intended for them were dropped into the bottom 7 ft. of the guides and left in. The bulkheads in the remaining six openings were removed after the concrete had set, in order to give a smooth clean sill for the flap-gate to butt against. This work was started on November 8th, 1913, finished on November 24th, 1913, and served two good purposes. It was done primarily as a precautionary measure; the bottom of these tunnels was very close to the natural bed of the river, from 12 to 18 in. on most of them, and with such a small sill for gates of the flap type to strike against, there was a chance for a stone or sunken timber to lodge against it at the time of closing and give a lot of trouble by preventing the gates from seating properly. With this additional 4 ft., the bottom of each tunnel was brought well above the bottom of the river, and also well above the tail-water elevation. There were two gates to each opening, and of the eleven openings, five were of the stop-log type, shown by Fig. 15, and six were of the flap-gate type, Fig. 16. When all these eleven openings had been filled with from 4 to 6 ft. of concrete as described, the ten smaller 8 by 12-ft. gates, which were of the flap type, were hung and securely closed on December 21st, 1913, and caulked around the outside. No water was running through these openings at the time, their bottoms being only 4 ft. below the tops of the large openings. The closing of these gates was a very simple and easy matter, and they were caulked and inspected from the upper side. On December 21st also, five of the larger openings were closed with the ten stop-log gates. The closing of these gates was easily accomplished and did not raise the water very much, as only a small quantity was running through this particular set, the water stage being very low. These gates were then securely wedged and caulked on the outside, up-stream face, and the filling of the tunnels was started. Next, a light template of the larger type of flap-gates still to be closed was made, and each of the openings was carefully checked with this template to be sure that there were absolutely no obstructions to the closing of the gates

and that nothing of any kind would foul them while they were being lowered. This check was made several times and just previous to the closing.

Owing to the non-delivery of the penstock steel gates by the manufacturers, it was deemed necessary at this time to drop stop-log gates in front of each penstock opening, there being six penstocks with two gates each. As the time was very short and the weather was threatening, it was not possible to secure timbers large enough to withstand a maximum head, consequently, the logs were dropped into the gate grooves and reinforced by four 18-in. I-beams. Later, logs of the proper size were dropped into the grooves regularly provided, the I-beams and logs were removed, and the steel gates put in. The I-beams were those bought for carrying the spillway operating mechanism from pier to pier. All these stop-logs were in and properly bolted and caulked on December 27th. On December 28th, at 7 A. M., the large flap-gates remaining were lowered to a horizontal position, clearing the water by about 2 ft. and rigged up so that by attaching a loose line on one side of the cables holding it above and cutting another loop the gate would fall, but the cable would be held by the loose line and not catch under the gate as it swung shut. On December 27th, rain commenced, and the river, which had been rising gradually all the while, started to rise rapidly, and it was decided to close the remaining gates at once, everything else being in readiness. The gates were closed in the order and at the times shown in the following:

Gate No.	Time closed.
*12	11:20 A. M.
11	1:10 P. M.
10	1:13 "
9	1:18 "
8	1:22 "
7	1:26 "
6	1:28 "
5	1:47 "
4	1:51 "
3	1:54 "
2	1:57 "
1	2:00 "

* Gate No. 12 was first lowered in order to try out the method. The closing of the dam really started with Gate No. 11.

Each gate, after being cut loose, followed three distinct motions: First, a rapid fall of 2 ft. until it struck the water; second, a very slow swinging in with the current until within 3 ft. of closing; and third, a quick, abrupt slam against the concrete face of the wall. The leakage was very small after closing, and was taken care of as described later.

Before the final closure was made, all the gates in the wasteway section were examined carefully, and raised and lowered to test them out in the dry. Plate XXX shows this wasteway section. Two sets of these gates were left open, and water passed through them during the period of filling the reservoir and for some time afterward, serving to hold the water on the crest of the spillway a little lower while the tunnels were being finally concreted. The water in the reservoir rose with fair rapidity, due to heavy rains, and flowed over the crest of the dam at 9 A. M. of January 1st, 1914. During the period of filling, a careful watch was kept of the river bed below the dam and in the vicinity of the dam itself for any indications of leaks or springs. None whatever was seen, and the dam and rock were absolutely tight.

Concreting the Openings.—In order finally to fill the openings through the dam, the water pouring over the crest had to be diverted and kept out while the work was being done. This was accomplished by building six gates, 3 ft. high and 30 ft. long, reinforced on the back with hog rods, and dropping them into the gate slots provided for the spillway gates. There was only a depth of about 18 in. of water over the crest at this time. All leakage under these gates was stopped by dumping clay and cinder just outside of them, their seats being about 3 ft. from the front face of the dam. It was then necessary to take care of leakage through the openings below and devise a scheme for filling them effectually. The leakage was taken care of in the following manner: A water-tight bulkhead was built about 8 in. back from the inside face of the final closure gate, and a sufficient number of drainage pipes was run from this bulkhead to the down-stream face of the dam to take care of all the water, the leakage being taken care of in most openings by from two to four 3-in. pipes. These pipes were afterward filled with cement grout under a pressure of from 10 to 40 lb. per sq. in. In order to pour the concrete into the openings directly from the cableway, hoppers 6 ft. square were built around the top shaft of each tunnel, but only one shaft was used

for filling purposes, the second shaft, being of no practical value, was not used. The concrete was dumped directly from $2\frac{1}{2}$-cu. yd. buckets into these hoppers and discharged down the shafts into the tunnels, where it was handled by men with shovels to within from 3 to 6 ft. of the roof, when work was stopped and the concrete allowed to set for at least a day. In the first few openings filled, it was thought best to build a concrete wall about 6 ft. thick of a medium dry concrete, to tamp it thoroughly, and to get a tight positive seal. As the wall did not prove to be water-tight, the method was discontinued. The method finally adopted for securing an absolutely tight seal was, as already partly described, to fill the tunnel throughout its whole length to within from 3 to 6 ft. of the roof, the men carefully spading and working this concrete with shovels so as to make a tight job. After this had set for 24 hours, it was carefully washed and cleaned and then a good flowing gravel concrete was dumped into the shaft, vent pipes having been properly placed, which vents also acted as tell-tales as the filling progressed. The back form was bolted up tight to the curve of the dam, and it was found that the dumping of this concrete down the main shaft would force concrete up and out of the back shaft to a height of 35 ft. or more above the top of the tunnel. The writers are of the opinion that, with a pressure of this kind and with a free-flowing concrete, every corner of the tunnel was bound to be tight, especially after the above described demonstration. As a further safeguard, however, a system of grout pipes was put in, and afterward grout was forced into these to fill any possible cracks.

In the ten small openings on the east side of the river, the same general scheme of filling was followed out, that is, a water-tight bulkhead was built 8 in. from the gate, leakage was carried off by pipes to the down-stream side of the dam, and concrete was poured to within 3 ft. of the top and allowed to set hard. Two 2-in. grout pipes were used, one on each side of the tunnel, and as close to the top as practicable, but below each keyway a T had been placed in the pipes and a nipple used so that its end would get within $\frac{1}{2}$ in. of the top of the keyway. Very little grout was placed in these pipes; in fact, concrete in nearly all cases completely filled the tunnels.

One difficulty, which should be guarded against in similar operations, was experienced in filling these tunnels: If the sand used was very coarse, as it was in one or two instances on this work, the concrete

would fail to run, and the tunnel would block itself. The remedy adopted by the writers in this emergency was immediately to dump 2-yd. batches of fairly rich mortar until the whole mass started moving again. As a rule, this remedy was satisfactory, and as soon as the mass started moving it came with a rush. In the worst case encountered, twenty-two 2-yd. batches of mortar were used to seal the top effectually. After grouting and setting a proper length of time, the openings were all tight, and showed very small negligible leaks which shortly clogged themselves.

SECTION D—GENERAL ORGANIZATION OF THE CONTRACTOR AND POWER COMPANY ON THE WORK.

General Organization of Contractor's Forces.—The main features of the contractor's organization were as follows: A General Superintendent, with headquarters at the dam, was directly responsible to the New York office of MacArthur Brothers Company. He was also responsible for the execution of the contract, and worked directly with the Power Company's Resident Engineer on the job.

His assistants were: Superintendent of the Dam, Superintendent of Railroad, Superintendent of Quarries, Office Manager, and Master Mechanic. All other parts of the organization were handled under these general departments, each department being complete within itself, yet working closely with all the others.

The Superintendent of the Dam had charge of all day and night forces at work at the dam, except such as came under the Office Manager and Master Mechanic. He had a Night Superintendent, and, on both shifts, walking bosses and foremen in all the various departments of the work.

The Master Mechanic at the dam looked after the proper maintenance and operation of all machinery and plant, and all derrick runners, engineers, firemen, machinists, helpers, blacksmiths, etc., were under him. This department worked in very close relation with the Superintendent of the Dam, and derrick runners, cableway runners, pump men, etc., worked to a large extent under both departments. This relation between the Superintendent of the Dam and the Master Mechanic is not in general a wise one; in order to get the best results, the Master Mechanic should be a subordinate of the Superintendent of the Dam; however, in the case in hand, the scheme

worked satisfactorily. The machine shop was about $\frac{1}{8}$ mile from the dam, and a large part of the Master Mechanic's time was spent there.

The Superintendent of the Railroad was in complete charge of all railroad operations outside of the Lock 12 and quarry yards. He had a force of trainmen, yardmen, trackmen, dispatchers, etc., under him, and was held responsible for the movement of all material over the railroad. His headquarters were at the junction of the main line and the quarry spur, 7 miles from the dam, known as Camp No. 4.

The Superintendent of the Quarry had charge of all quarrying and crushing operations at Zion and Ellison Quarries, and the camps at these two places. He had day and night forces, with walking bosses, foremen, a master mechanic, timekeeping and clerical force, commissary force, and all the skilled mechanics, engineers, etc., necessary for the work.

The Office Manager handled all commissary matters, purchases of supplies, timekeeping, bookkeeping and all financial matters. He had at Lock 12 a general office force of bookkeepers, timekeepers, stenographers, clerks, etc. All commissary managers at the various camps came under his jurisdiction, as well as timekeepers and material checkers there.

Commissary Department: Organization and Methods.—The Commissary Department deserves special mention, as it was a most important part of the work. Supplies for all four camps and for about 2 000 men were handled through this Department, all kitchens, cooks, as well as stores, coming directly or indirectly under it. The store at Lock 12 was not very well situated, and should have been larger. The writers believe that, had a little more thought been spent on its planning, it would have brought in better returns than it did. No elaborate stock was carried, and no particular attempt was made to realize profits from merchandising. The writers feel confident that it would pay the expenses of all commissary forces to run a store of this kind along proper lines. A store operated just outside of the limits carried a stock of merchandise and fancy groceries, and, even though big prices were charged, a good profit was made by the owner. The writers would recommend that, on a job of this size, the commissary store be a large airy building, centrally located, have regular counters, shelves, and tables for the display of goods; it should have a butcher shop as one department, and also a cold drink stand. It is to be

noted that the small cold drink side line operated in connection with the commissary practically paid the wages of two clerks alone in the summer, and one gave only about half his time to it. One general commissary inspector should be appointed, and he should be experienced in operating a store of this kind. All the various camp commissaries should be under him, and it should be his duty to inspect and watch constantly the character and class of goods demanded and to fill that demand.

It is astonishing how negro laborers and others on construction work will spend all they make for good clothes, candy, fancy groceries, etc., and a good profit can be made from these. It should be the aim to make these commissaries pay all expenses in connection with them and make up the deficit, if any, in the boarding houses, thus relieving the job of that burden as much as possible.

General.—In general, this organization worked out well, and there seemed to be very little friction; at the height of the work all departments seemed to be imbued with a spirit of co-operation and a desire to make high records on the concrete work.

Labor.—The labor problem proved very troublesome. Negro labor was used almost entirely, although other classes were tried. The negro, with all his unreliable features, proved the most satisfactory, but had to be treated properly to get the best results. A regular labor agent was employed to handle exclusively the Power Company's work. His headquarters were at the dam, and he reported to the Contractor's General Superintendent. He was a man who knew the Southern labor market well and was thoroughly familiar with all conditions. The greater part of the negro labor came from Florida, and it was necessary to keep a constant stream of men coming all the time. Some negro labor was brought from Mississippi, Memphis, Tenn., and Birmingham, Ala. All transportation was paid by the Company, and deducted later from the laborer's wages. If he remained and worked 60 days, he made his transportation, and it was returned to him.

Quite a few foreign laborers were brought, *via* Savannah, Ga., by sea, from New York City, being secured by New York agencies, but they were not satisfactory at the dam. They were then tried at the quarry and did so much better that for a long period the quarry labor camp was almost exclusively of foreigners. However, the loss was so great on the transportation of foreigners from New York that

labor of this class was finally abandoned, and negroes were put in at the quarries on both day and night shifts. The skilled mechanics, hoist runners, machinists, etc., all came from similar jobs scattered all over the South, and were of the type who follow big jobs of this kind around the country. Quite a few carpenters were brought from New York City, principally Swedes, and proved far more satisfactory than the local talent. The local white labor, skilled and unskilled, was absolutely unreliable, and of an exceedingly poor class. This source did not afford any help whatever in solving the labor question, and was effective in only one section of the work, namely, in floating and placing the coffer-dam cribs. This class was entirely ignored, there being only a very few regular employees drawn from it.

General Organization of the Power Company.—Supervising the Contractor's organization on the power-house foundations and dam, the Power Company's organization was in general as follows: Representing the Chief Engineer directly was the Resident Engineer. His responsibilities under the type of contract used were really those of a General Superintendent of Construction rather than those of an Engineer alone, as not only was he held responsible for the proper execution of the plans, proper inspection of all materials in the field, and the mixing and placing, but also for the working of the job by the Contractor in a proper manner, the disposition of his forces, his expenditures, the proper running and sanitation of his camp, the handling of commissaries, the placing and lay-out of the plant, etc. Furthermore, the Resident Engineer was clothed with full powers to back up and force attention to all orders and instructions given by him in the conduct of the work.

Under the Resident Engineer were two Assistant Engineers, a Chief Inspector at the quarry, and an Inspector of the railroad; one Assistant Engineer looking after the details of the office and all the night work in conjunction with the Resident Engineer, and all such general details of the job as the Resident Engineer turned over to him for attention. The other Assistant Engineer, with his assistants, not only looked after the proper handling and placing of the concrete, but at all times kept close track of the Contractor's working organization on the dam to see that it was handling the work efficiently. He conferred constantly and worked almost entirely with the Contractor's Superintendent on the dam, his main duties being to look after the construction

on the dam and power-house only, to see that all features were according to the specifications, and that the forces were properly organized and handled.

A day and night inspection was maintained at the quarry, the Chief Inspector handling the day shift and the inspection work on that shift. He was assisted by a timechecker, clerk, and night inspector. He was held responsible for the proper inspection of all crushed and cyclopean stone, and saw that only clean materials were loaded. He watched the loading at the quarry, saw that all stone was washed before going to the crushers or to the dam as cyclopean, and measured all cars of crushed stone before they were pulled out. He also was held responsible for the proper checking of all materials and labor used by the contractor there, and reported any inefficient handling of men or equipment. The Night Inspector performed the same duties at night, reporting to the Chief Inspector. The Power Company's forces were provided with a comfortable house and office entirely separate from the Contractor's forces. They, however, took their meals at the Contractor's mess hall.

The Inspector of the Railroad performed the same general duties on the railroad as other inspectors, and his assistant also checked time and materials, as already outlined. His duties, however, were not as important as those of the other inspectors.

A force of time and material checkers, entirely independent of that of the Contractor, was maintained in each camp, and vouchers and pay-rolls were approved only on the Power Company's check. All expenditures for material and labor had to receive the approval, in writing, of the Resident Engineer, and this approval was essential to the passage of any voucher for payment.

General day and night forces of inspectors, checkers, timekeepers, etc., were maintained by the Power Company at the dam. All the day inspection work was performed under a Chief Inspector, and he, with his inspectors at the dam and at the mixer, checked all forms, looked after the proper mixing, placing, and working of concrete, the proper preparation of the surface of old concrete, saw that all bolts and reinforcing were properly set, etc. The inspectors on the dam and power-house gave orders to the concrete foremen and walking bosses where they concerned the proper working of the concrete and the cleaning of the surface of old concrete before placing new concrete

on it. They had the authority to prevent any of the Contractor's men from commencing to place concrete if the forms were improperly cleaned out. Great stress was laid at all times on the proper cleaning of the concrete. The mixer inspectors watched the proper proportioning of the mix and the quantity of water necessary, receiving their instructions from the inspectors on the dam. At the mixing plant, there was also a cement checker on each shift.

The night shift organization was handled in the same manner, with an Assistant Engineer in general charge for the Power Company, and a Chief Night Inspector under him. All the survey and layout work was handled by two small parties of two or three men each. One of these parties was assigned to the dam and another to the power-house. All levels, lines, grades, etc., were given by these parties. Before and after forms were set up by the carpenters, the lines and levels were given by one of these field parties, and the inspectors would allow no concreting in a form until it was "O. K'd". as to grade and line by the Chief of the Party.

This system of handling the inspection and survey work proved most satisfactory, and is recommended for similar work.

Power-House Superstructure and Equipment.

As previously mentioned, the construction of the power-house superstructure and the installation of machinery was done by the Company's own forces. In October, 1913, the organization of the Company's construction forces was begun, most of the men being taken from Gadsden, where, as noted early in the paper, a 10 000-kw., steam turbo-generating station had been constructed.

A camp was constructed near the main camp of the Contractor's forces, and independent mess halls were provided. Storage yards for structural steel, brick, machinery, etc., were constructed part way down the Low-Level Line, and a traveling gantry crane, later erected in permanent position on the up-stream side of the power-house, was placed in these yards for the purpose of unloading and storing heavy machinery parts.

Steelwork.—On November 4th, the foundations were in such shape in the power-house that steel erection was started. A spur track was built from the Low-Level Line in the bottom, entering the west bay of the power-house on a trestle at Elevation 370, approximately, it being

planned to bring all steelwork, machinery, etc., in on flat cars here and remove it with first a locomotive crane and later with the permanent 100-ton crane. A 30-ton locomotive crane was set up inside the power-house for the purpose of erecting all columns and cross-girders. This crane had a 50-ft. boom with a 30-ft. extension. On Figs. 45 and 46 this crane is shown at work erecting columns, and also the track on which all steel and other materials were brought in. As soon as the steel was in condition to receive the 100-ton crane, it was erected, and the locomotive crane was withdrawn for the purpose of loading cars in the yards. A traveling stiff-leg derrick was then erected on the heavy cross-girders above the 100-ton crane, and it handled the remainder of the steel erection, the 100-ton crane taking the steel from the cars below and delivering it to points under the traveling derrick.

Brickwork.—After riveting was sufficiently advanced, the brickwork was started on a previously moulded concrete water-table. All the window sills, cornices, and ornamental work were of concrete blocks, moulded and cast on the work. All concrete block forms were of cast iron, and the results obtained were very good. Gravel and coarse sand were used for making the concrete. They were mixed fairly dry, being well packed and rammed into the moulds.

Erection of Units.—The foundation rings and shell of the units came in sections. These sections were loaded by the 30-ton locomotive crane on flats and delivered to the 100-ton crane in the power-house. These sections were then assembled on the floor adjoining the foundation on which they were to be set, and put in place, properly lined and leveled up, grouted, and then concreted. The type of speed ring and casing used permitted it to be set directly on the foundation previously prepared. It was blocked into permanent position by jacks and holding-down, foundation blocks, and then solidly concreted. The first speed ring was set for No. 4 Unit on January 8th, 1914, and this unit went into commercial service on April 12th, 1914, the others following rapidly thereafter. The first unit was operated through a temporary switch-board and transformer-house set up outside the main building, in order to start selling this power at the earliest possible moment. This allowed the setting up complete of the main busses, switch-boards, and wiring, without current being on them. Later, when these three units were all installed and everything in readiness, the unit that

had been operating temporarily was pulled out of service, the transformers were put in their permanent locations, and the few connections between the unit and switch-board easily and quickly made.

Gates.—Penstock gates were set at the time the units were being prepared for service. Some trouble was experienced on all gate frames on account of the castings being too light and warping or springing. This was true of the spillway gates and the guides for the screens in front of the penstock gates. It is almost impossible to block these to exact line, and, in the cases in question, the clearances were entirely too small. Also, the method of setting gate frames in the original pour of concrete around them should be avoided. In view of the experience gained on this work the writers recommend strongly that in similar designs the following points be observed in the design and erection of cast or structural gate frames:

1. See that ample clearances are allowed, giving wide bearing faces to make up the excess width allowed.
2. All faces should be machined true and square, and should be carefully inspected with that end in view at the mill or factory.
3. Grooves and pockets with proper anchorages should be left in the concrete, the frames should be set up after the mass of concrete is poured, and then carefully lined, anchored, and grouted in place.

A gantry crane which traveled on the front side of the power-house, at Elevation 425, handled and set all the large penstock gates. These gates weighed about 12 tons each, and were operated by large cylinders, using oil under pressure. The spillway gates were all assembled and riveted in the assembly yard, some distance from the dam. They were then loaded on cars and delivered to a scow, built for the purpose, just above the dam. They were towed to the front of the openings, and put in place. These gates were operated by a traveling car which, through a flexible coupling, engaged a gear operating the hoisting mechanism of each gate. This traveling car was in duplicate, one electrically operated, the other steam-driven.

Reservoir Clearing.

Of the 4 700 acres of reservoir land, 1 700 acres were more or less heavily wooded. A thorough investigation showed that there would

be no actual ill effects to the health of the community at large, by leaving this timber standing, but the temperament of the hill people residing in the neighboring territory, and their respect for the law, in so far as it may assist them in obtaining compensation for imagined damages, made it appear prudent to clear the reservoir. This decision was strengthened by the fact that a power company in Georgia, which had not cleared its reservoir, was having endless legal trouble from damage suits filed against it. After consultation with the health officials of the State, it was decided that no timber should be left standing, or in a fallen position, between the 420-ft. contour, the normal lake level, and the 410-ft. contour, the limit of the draw-down. Wherever merchantable timber was within a reasonable distance of the railroads, it was sold, and a quantity of pine timber was converted into charcoal; whatever could not be disposed of in either way was burned on the ground, if possible.

Transmission Lines.

The standard line construction adopted by the Company involved the transmission of current at 110 000 volts and distribution at 22 000 volts. For the former, the double-circuit, galvanized-steel towers were adopted. These towers carry two ground wires of ⅜-in. Siemens Martin, and six No. 00 copper, 7-strand conductors, drawn to 55 000 lb. per sq. in. in tensile strength. The towers were of three types: Type A used in rough country, weighs 4 600 lb., is 68 ft. high, and has a 10-ft. spacing of conductors vertically and 15-ft. horizontally on the top and lowest arms, and 16 ft. 6-in. on the intermediate arm. The base of this tower is 15 ft. square. Type B towers were used as strain towers; they weigh 4 700 lb. and are of the same general design as the other types except that they have bases 17 ft. square; these towers are used at angles between 3° and 35° and at every 2½ miles on the lines. The Type C tower is used in flat country in connection with Type B towers, the latter being placed at each mile point. These towers weigh 3 600 lb., are 65 ft. high, and have bases 16 ft. square. The ground clearance allowed was 25 ft., and the towers were located so that with a ½-in. coating of ice and 8 lb. wind pressure, this clearance would be maintained. The normal spacing for Type A towers is 700 ft., and for Type C, 600 ft.; the maximum allowed spacing for Type A is 1 200 ft., and for Type C, 900 ft.

The insulators used on the 110 000-volt lines were of three makes: the Ohio Brass, Thomas, and Locke. The strings were made up of 6 units on suspension and 7 on strain. The disks are 10 and 12 in. in diameter. The 6-disk strings were tested to 350 000 volts dry and to 255 000 volts wet, before flash over. The test for mechanical strength consisted of a pull of 5 000 lb. applied in the direction of the axis.

The right of way secured for transmission lines was 100 ft. wide. The towers were set on the middle of this strip, and, 10 ft. from the edge, there was carried an all-copper telephone circuit of No. 8 hard-drawn wire, on 22-ft. creosoted poles, 175 ft. apart. On the telephone lines, porcelain insulators, designed for 6 600 volts, were used on a single cross-arm. The line is transposed every fifth pole.

Wagons were especially designed for the transmission line surveys, and their equipment was very compact. Each wagon contained sleeping accommodations, and cooking and table utensils for a party of thirteen men. A negro cook and a negro who acted as driver and general utility man were carried with the party.

The sequence of construction was as follows: When the right of way was secured, a clearing gang, normally consisting of sixty men and two foremen, cleared the line. Immediately following the clearing gang was the telephone gang, consisting of a foreman and twelve men. This gang erected poles and strung the line complete, tying into the public long distance telephone lines at convenient points, so that at all times there was easy communication with the main office. Following a short distance behind the telephone gang was the foundation gang of seventy-five men and three foremen. This gang dug holes, set the stubs of the towers, and back-filled, or set the rock-anchors, as the case might be. The assembling and erecting gangs of forty men total and two foremen for the two gangs, followed as closely as convenient. The stringing gang, of sixty men and three foremen, hung insulators, and strung the ground wires and conductors, finishing the line ready for service.

Fig. 50 shows part of the stringing gang at work on the line from Lock 12 to Anniston, the right of way being occupied by the tower line and a pole line carrying 22 000 volts. Fig. 51 shows the Sylacauga-Alexander City, 22 000-volt line. In this line, the conductors were carried on wishbone-type cross-arms, and the ground wire on

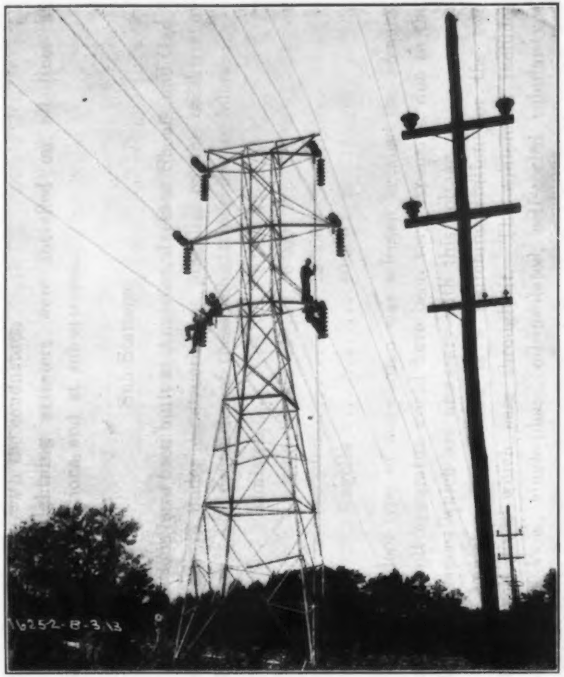


FIG. 50.—STRINGING COPPER CONDUCTORS ON TRANSMISSION LINE OF 110 000 VOLTS.

FIG. 51.—DISTRIBUTION LINE FOR 2 000 VOLTS, USING WISHBONE TYPE OF CROSS-ARM.

Fig. 11.—Intermediary Line for 2,300 Volts. Using Wishbone Type of Cross-Arm

Fig. 10.—Suspension Chain Insulators on Transmission Line of 110,000 Volts

a bayonet. On all 22 000-volt lines, the telephone line was carried on the same pole with the conductors.

Electrolytic lightning arresters were installed on all lines at the generating stations and at sub-stations.

Sub-Stations.

Sub-stations have been built at Anniston, Jackson Shoals, and Gadsden, and one is under construction at Magella, on the outskirts of Birmingham. The capacities of these sub-stations are as follows:

Anniston	6 000 kv-a.
Jackson Shoals	6 000 "
Gadsden	12 500 "
Magella	40 500 "

The outdoor-type of sub-station was selected because the housing of 110 000-volt apparatus would have been very expensive, due to the large clearances which are necessary with this voltage.

The Anniston Sub-station is a switching station for the two 110 000-volt lines which pass through it. The equipment includes three 2 000-kv-a., single-phase, oil-insulated, self-cooled tubular-type transformers, stepping the voltage down from 110 000 to 22 000 for distribution in the Anniston district. The efficiencies of these transformers at full, three-quarters, and one-half load, are 98.4%, 98.2%, and 97.7%, respectively. A small brick building, Fig. 53, houses the switchboard, three 667-kv-a., 22 000 to 2 300 transformers for local duty distribution, the storage batteries for operating the oil-switches, and a motor generator set for supplying direct current to the Anniston Street Railway.

The Jackson Shoals Sub-station is a switching station for two 110-kv. from the north and south and one 110-kv. from the west. At this point, the voltage is stepped down to 22 000 for distributing lines to the north, south, and west. The equipment includes three 2 000-kv-a., oil-insulated, water-cooled, single-phase transformers, with efficiencies of 98.05%, 97.9%, and 97.5%, at full, three-quarters, and one-half load, respectively. Near the sub-station is the small hydro-electric plant generating at 2 300 volts. A bank of transformers steps this voltage up to 22 000 volts, and the plant is tied into the 22 000-volt bus of the sub-station.

The Gadsden Sub-station is a distributing as well as a step-up station. The generating voltage of the steam plant is stepped up from 2 300 volts to 110 000 or 22 000 volts, or the 110 000 voltage of the transmission lines is stepped down to 22 000, as the case may be. The equipment of the station includes two banks of three transformers each, and a spare-all single-phase, oil-insulated, and water-cooled—each of 2 100-kv-a. capacity. These transformers have three windings, for 2 300, 22 000, and 110 000 volts. The guaranteed efficiencies at full, three-quarters, and one-half load are 98.3%, 98.1%, and 97.6%, respectively.

The Magella Sub-station is a distributing station for three 110 000-volt lines, two from the south and one from the east. The station will step down from the 110 000 volts of the transmission lines to 22 000 volts and to 13 200 volts for distribution, the latter being for distribution into Birmingham. Provision has been made for increasing the capacity of this station from 40 000 to 67 000 kv-a. The equipment includes three banks of three transformers each, and a spare-all, single-phase, 4 500-kv-a. capacity, oil-insulated, and water-cooled.

Fig. 52 shows the bus structure of the sub-station at Anniston, which is typical of the others. It is built of towers and girders, this construction having been found to be more economical than an arrangement of towers alone. All the material in the structure is galvanized steel. The weights of the structures are: Anniston, 22 tons; Jackson Shoals, 35 tons; Gadsden, 30 tons; and Magella, 50 tons.

The 110 000-volt busses are No. 00, B. & S. gauge, copper wires, supported horizontally by seven, and vertically by six, 10-in. disk insulators. The buses are 8 ft. apart and are 5 ft. from the nearest steel.

The oil-switches are of the out-door, solenoid-operated, remote-control type, operated from storage batteries in the switch-houses.

Provisions are made at each sub-station for the repair of transformers. There are electrolytic lightning arresters on all 110 000 and 22 000-volt lines, as a protection against the severe electric storms of the region.

It is proposed to standardize future installations of sub-stations into capacities of 3 000, 6 000, and 10 000 kv-a. The bus structure, being made up in unit panels, will be adapted readily to this standardization.

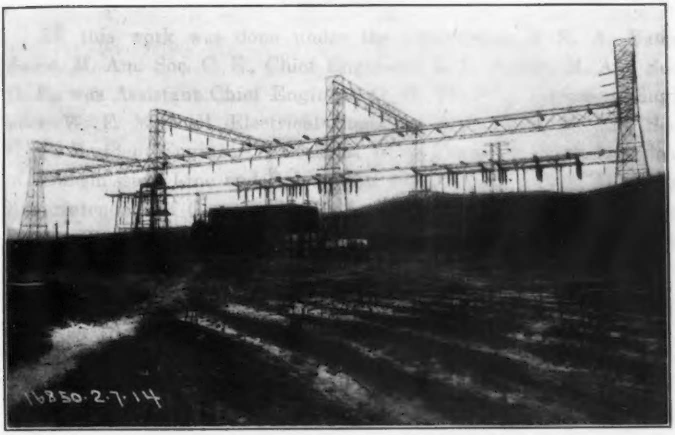


FIG. 52.—OUTDOOR SUB-STATION AT ANNISTON, ALA.; CONSTRUCTION IS TYPICAL OF ALL SUB-STATIONS.

FIG. 53.—INTERIOR OF SWITCH-HOUSE AT ANNISTON SUB-STATION.

Organization.

All this work was done under the supervision of E. A. Yates, Assoc. M. Am. Soc. C. E., Chief Engineer; E. L. Sayers, M. Am. Soc. C. E., was Assistant Chief Engineer, O. G. Thurlow, Designing Engineer, W. E. Mitchell, Electrical Engineer, A. C. Polk, M. Am. Soc. C. E., Resident Engineer at Lock 12, W. C. Cram, Jr., Superintendent of Transmission Line and Sub-station construction, and S. B. Jones, Superintendent of Construction at Lock 12 Power-House and Gadsden Steam Plant. MacArthur Brothers Company were the Contractors for the power-house foundations and dam.

DISCUSSION

Mr. Harlow.

James H. Harlow,* M. Am. Soc. C. E. (by letter).—Like most writers of technical papers, the authors fail to describe the plant constructed so as to permit the reader to answer the question "Will it pay?" This is the question uppermost in the minds of those to whom we go for funds with which to carry out our ideas. It involves two other questions, *viz.*, the cost of the plant under consideration, and the character and extent of the market for the power to be developed.

The writer finds one stock objection to the construction of a hydro-electric plant, and that is, "a steam plant can be built for much less, and the best turbo-generators can be operated on 20 lb. of steam per kw-hr.". Several times when this statement has been investigated, it has been found that 20 lb. of steam did not include the auxiliaries, and instead of 20 lb., it was nearer 30 lb.

The writer is interested in several hydro-electric plants, and, when the stream will give power for 9 months per year and for the other 3 months the plant is operated by steam, the resulting power is produced more cheaply than by steam alone.

A year has passed since the construction of the Lock 12 Development, and some interesting facts must have accumulated in the operation of the plant. The authors will supplement the paper, perhaps, by answering the question, "Has it paid?"

What was the cost (ready to operate) of the hydro-electric plant per kilowatt on the station switch-board?

What was the cost of the steam auxiliary plant per kilowatt on the station switch-board?

(The cost per kilowatt should be based on the actual annual output of each plant.)

What was the actual output of each on the switch-board?

The greater the detail of the costs, the more value it will afford to designing and operating engineers.

Another point of interest is the size of the units. The authors say, "The adoption of this size of installation was dependent on many factors and a number of assumptions, * * *." May we hear further on this point later?

The writer has knowledge of a plant designed to adopt units of 37 500 kw. Are we not trying to follow a fetich in these large units?

From some of the writer's investigations, there is a point in the cost of generators where the cost per unit of output rises. Is not the cost of the combined hydraulic and electric machinery a rather small part of the total development cost, and should it be followed as far as we seem inclined?

* Darlington, Md.

Mr. Harlow.

At the plant in question, can more than one unit be operated at the extreme low-water flow, and if two must be run to get the use of all the water flowing, is not the efficiency being reduced much below that expected?

The highest efficiency is in demand at times of low water; when there is plenty of water, efficiency is not so necessary. The writer has demanded that the highest efficiency shall be at about from 75 to 80% of gate-opening. Turbines cannot be well regulated when operating at near full gate.

How are the authors going to determine whether or not the "guaranteed efficiency" is obtained? They certainly do not depend on the Holyoke tests of a (comparatively) small turbine and then the guess of the builder that the full-sized turbine will "fill the bill".

The best hydraulic tests are probably not within several per cent. of the truth. Do we know enough about the "chemical test" to be sure we know the exact efficiency?

Horizontal vs. Vertical Units.—Unless the tail-water rise is in excess of the suction effect, why was the vertical unit adopted instead of the horizontal?

Some years ago we required our steam turbo-generators to swing on a vertical axis, but now we are going to the horizontal.

The hydro-generator is now going toward the vertical; shall we, after a while, swing back to the horizontal?

The McCall's Ferry plant has been referred to as using the vertical type, but here the tail-water rise may be as much as 35 ft., and it must be remembered that the third thrust-bearing is being tried, and successfully thus far; but it took the third trial to get the bearing. A longer trial may change our views.

Papers like this are valuable, but more attention should be given to "will it pay" or "does it pay", and to do this we must have details of the costs as well as of the construction.

Mr. Coombs.

R. D. COOMBS,* M. AM. SOC. C. E.—As it is the avowed purpose of the authors to bring out discussion on the latest ideas and methods relating to hydro-electric power plants, including those pertaining to transmission line construction, the speaker will confine himself to a discussion of, and perhaps some additions to, the methods described in the paper.

Unfortunately, the authors have followed the usual practice and dismissed the subject of line construction in a very sketchy way. The word "unfortunate" is used advisedly, as the subject merits more consideration.

Apart from the real need of more literature on this subject, in order that future work may have the benefit of past experience, it

* New York City.

Mr. Coombs. would seem that the sole link connecting the expensive power-generating station with the consumer should be considered of relatively greater importance.

In addition to the two foregoing considerations, the speaker believes that the comparative importance of the line, from an economic standpoint, has usually been partly disregarded. As the entire installation is a commercial enterprise, the income of which is derived from the sale of the current carried by the transmission lines to the purchasers, the lines themselves may be a large factor in the possible quantity of unproductive, expense-bearing time. In other words, the connecting link may cause stoppages in the sale of current, and, in extreme cases, render the entire installation unproductive for various periods of time. Therefore, the maximum possible security from interruptions to service, obtainable with a reasonable expenditure, is very desirable in any line installation.

The transmission lines—and in this discussion no special distinction will be made between so-called transmission and distribution lines—may be of two general classes: aerial open wires and underground insulated wires or duct lines. The latter are approximately three times as expensive to build as the former, and as they have but few applications in hydro-electric development in the United States may be omitted from consideration herein.

In reviewing the data in the paper, it will be noted that in general the design appears to follow the latest methods of wide-base tower construction. These are—to give them in outline—long spans, stranded cable, overhead ground wires, vertical spacing, and staggered middle cross-arms. Unfortunately, information is lacking as to the details of construction, and the reasons for various decisions as to such details.

Referring to the lengths given for the two kinds of cross-arms, it would seem that there must be an error in the stated length of the middle arm, because, as given, it only offsets the wires 9 in. This small offset, which is introduced in such lines in order to minimize the probability of wires sagging, falling, or blowing into contact with each other, is inadequate for the purpose.

It is stated that the Type A towers, which are 68 ft. high, were used in rough country with standard spans of 700 ft., and that Type C towers, 65 ft. high, were used in flat country, with standard spans of 600 ft. Unless the towers could be located in an almost miraculously favorable manner, the speaker believes that this relation would not obtain, because rough country ordinarily compels the use of spans shorter than standard, unless the towers are made higher. This is due to the constant need of decreasing the sag in order to maintain the required clearance of the wires above the ground.

FIG. 54.—TWO-CIRCUIT STEEL POLE LINE.

Although not thus stated, the inference may be made that the design, as well as the clearance, was based on a loading of ¼ in. of ice and 8 lb. of wind pressure. The speaker believes that a loading of ½ in. of ice and a wind pressure of less than 8 lb. is better general practice. Some reduction from the latter loading may be justified in certain cases, particularly in construction with very long spans, but the speaker believes that in actual practice it will be the wind pressure, rather than the ice, which will be reduced. However, in the case in question, it may perhaps be argued that a large accumulation of sleet need not be expected on 110 000-volt wires while alive, and that the period during which the wires will be dead would reduce the probability of the maximum load to a negligible quantity. It would be of interest if further explanation were given in regard to the relative strength of the towers in the standard spans, and in the maximum spans mentioned. Ordinarily, such maximum spans are of rare occurrence, and they are frequently guyed in order to withstand the heavier loads. **Mr. Coombs.**

Although no particular criticism may justly be made of the main transmission line, as it fairly represents one of the two best types of construction, there may be some difference of opinion as to its relative advantage as compared with two one-circuit pole or tower lines. The so-called distribution line, of 22 000 volts, on wood poles, however, is not of as excellent construction, and, if its location is permanent, a steel pole line would perhaps have been justified instead.

As steel pole lines, such as those shown by Fig. 54, with spans of medium length, are growing in favor for 22 000-volt transmission lines, they are worthy of consideration for a 22 000-volt distribution line. The size of the conductors in the 22 000-volt line is not stated, and a photograph is an uncertain basis for measurement, but the pole spacing of 175 ft. and the vertical and horizontal spacing of the conductors do not appear to be consistent.

(By letter.)*—In reply to Mr. Mitchell's comments on the writer's previous discussion, it would seem to be in order to call attention to a somewhat popular fallacy, that is, that the design and construction of transmission lines is primarily electrical, rather than structural. It is true that the insulation and the electrical capacity of the conductors are purely electrical matters, but it would appear to the writer that all the rest of the work belongs to the department of civil engineering, and that it would have been in order for the authors to have included a complete description of these features in a paper before a civil engineering society.

The foregoing remarks are made in no spirit of criticism, but rather as a request that such matters should more frequently be included in papers before this Society. The writer has no knowledge

* This was presented after Mr. Mitchell's discussion had been published.

Mr. Coombs. of the details of the construction in question, other than those furnished by the paper and Mr. Mitchell's discussion, and although from the data at hand it would appear that the line was carefully considered and well designed, the information given is subject to misinterpretation because of its lack of elaboration.

The use of several routes, instead of combining all the circuits on a single right of way, is excellent practice, and might well be followed in other instances.

Mr. Mitchell has apparently misunderstood the writer's reference to the use of long spans in rough country. The suggestion was not that an "average" span of 700 ft. was not used with the Type A towers, but that if the "standard" span on a Type A tower were 700 ft., it would be improbable that the average span with those towers in rough country would be 700 ft. In other words, there was a difference in height of only 3 ft. between the Type A and Type C towers, and it was apparently stated that the standard span of the former was 700 ft., and of the latter 600 ft. There is quite a difference practically between the meaning of the words "average span" and "standard span", the former being the final result, and the latter the original design, which provides a theoretical clearance above the ground in flat country. Therefore, unless the towers can occupy hill tops, or an extra clearance is allowed in the design, it frequently happens that intervening elevations, or the loss of clearance on hillsides, materially decrease the actual span length.

In view of the fact that a careful investigation was made of the climatic conditions, the writer's previous suggestion, advocating a heavier sleet load, is perhaps not necessary. It will be found, however, that sleet may be encountered, during the probable life of a well-built transmission line, in a great many sections throughout America, even in the South. There may be some question, moreover, as to the desirability of providing a sag corresponding to the standard sleet load, even in non-sleet regions, because such sags decrease both the normal and the maximum loads on the line, and, in general, produce a line which is well able to distribute and equalize excess stresses.

Referring to the additional information furnished by Mr. Mitchell in regard to the wooden pole distribution lines: The use of properly creosoted timber, of course, vitiates somewhat the writer's previous criticism; but he does not consider the use of flexible towers (poles?) as being equivalent to the semi-flexible steel pole suggested.

As previously stated by the writer, "a photograph is an uncertain basis for measurement", and he still believes that he was correct in saying that the appearance of the wood pole shown in Fig. 50 would not indicate its true dimensions. The vertical and horizontal spacings appear to be equal, and, if it followed the usual dimensions of a 22 000-

volt line, it would not provide nearly as great clearances as stated by Mr. Mitchell, which, of course, are entirely adequate. Mr. Coombs.

Hugh L. Cooper,* M. Am. Soc. C. E.—The cross-section of the wheel setting, Fig. 11, shows that the thrust-bearing is carried on top of the generator frame. It would be interesting to know the experience with the thrust-bearings at this plant, particularly with reference to their ability to carry the load without interruption. The speaker has been informed recently that, in operating under conditions approximating full load, these bearings have given trouble. Mr. Cooper.

These Kingsbury bearings have been used with much success in the plant of the Mississippi River Power Company, at Keokuk, and the speaker believes that they are, undoubtedly, the coming bearings for vertical shafts with heavy loads, provided they are supported properly. Exact data regarding any trouble with this bearing at the Coosa plant would be valuable. On more than one occasion the proper location of the thrust-bearing on the shaft has been under discussion. Some have been of the opinion that such bearings can be supported on long metal spans and run successfully; others hold that they should never be placed in that way, but should be supported on a non-elastic foundation—the firmer the better. Specific and correct data on this subject will be of much value in designs for future work of this kind.

In reference to the pit liner, Fig. 13, will the authors state whether or not the castings were machined after they were set in concrete? The speaker has always found that the shrinkage of the concrete threw the casting out of line, no matter how well it was milled in the shop. On the work at Keokuk the pit liners were set with the greatest possible care, using specially designed leveling jacks, and notwithstanding the great weight and stiffness of these liners, there was a considerable change in the alignment after the concrete had set. This could not be neglected, and therefore the main bearing surfaces were milled *in situ*.

Heinrich Homberger,† M. Am. Soc. C. E. (by letter).—This very interesting paper is without doubt one of the most valuable additions to the literature on large, low-head, hydro-electric developments of the most recent type. Mr. Homberger.

In reference to the turbines, the authors state that one of their features is that they are "self-contained." With first-class design, vertical-shaft turbines of this particular type, of such huge dimensions, could hardly be otherwise; each, of necessity, must be a homogeneous structure, or it would be impossible to make a satisfactory job of the installation in the plant. It is hardly possible to believe that

* New York City.

† Mill Valley, Cal.

Mr. Homberger. any modern manufacturer would contemplate building such a turbine without setting it up completely in the shop before shipment to the plant, and, in order to clear up what the authors had in their minds, it would be interesting to note what character of "very expensive machine work is commonly done in the field" with turbines of the size and type described.

The turbine runner is fastened to the shaft with a taper fit. This is very expensive work if thoroughly well executed. A taper fit has the advantage that the joint may be readily broken, which is of importance where the hub is to be removed from the shaft frequently. This is hardly expected to be the case with a turbine runner. If a parallel press fit is selected instead, it has the advantage that the runner can be driven home to a fixed point and its location is absolutely determined. Of course, there is no leeway in the location of the runner on the shaft, and to make the taper fit a success, it is generally necessary to press the runner on and off the shaft many times, with intermediate scrapings, until the hub is exactly in its right position on the shaft and at the same time comes to a perfect bearing thereon. It seems that the taper fit is rather a design borrowed from marine practice and used for fastening a propeller to the tail-shaft. Propellers are likely to have to be replaced much more often than turbine runners, and the exact location in the axial direction of the shaft is of no importance. It is sufficient to scrape the propeller hub to a fit on the shaft and then tighten the nut.

With reference to the efficiencies which have been guaranteed: Supposing that the generator was expected to operate on a 75% power factor, this would allow about 25% over-load capacity in the turbine. It is common practice to select the point of highest efficiency at about three-quarters of the capacity of the turbine, which would be somewhat below the rated capacity of the generator, because, at a plant with several units, the machines are expected to operate normally at about the rated capacity of the generator. With the Alabama turbines, the highest point of efficiency appears to have been very near the full capacity of the turbine (the exact point is not given), and it is very rarely expected that the turbine will operate under such a heavy load; in reality, it would probably be operated mostly near 14 000 h.p. output where the efficiency is 4% lower than the guaranteed maximum.

The question of the value of Holyoke tests compared with field tests after installation has been thrashed out frequently, and the writer was pleased to see how much stress the authors have laid on the necessity of checking up the Holyoke results by actual field tests. Although Holyoke tests are very interesting to the designer of turbines, they give an insight only into the characteristics of the turbine runner

and its behavior under varying conditions; all the other accessories which make up the complete turbine, and the general mechanical and hydraulic conditions of the setting, which greatly influence the efficiency, cannot receive any consideration at the testing flume. Mr. Homberger.

The question whether preference should be given to vertical or horizontal units has been argued, and the selection of the former is explained by stating that horizontal units would promise not as high an efficiency on account of the necessity of sharp bends in the draft-chest, causing loss of head, and that there were other losses from the confluence of streams in the draft-tubes of multi-runner horizontal units.

To the writer, it does not seem necessary to make the curvature of the draft-chest of a single-discharge horizontal turbine of any smaller radius than the draft-chest of a vertical unit. Referring to Fig. 7, which shows the draft-elbow as constructed in concrete masonry, exactly the same curvature could have been chosen for a single-runner, single-discharge turbine with horizontal shaft. As far as space is concerned, if the units were arranged athwart the power-house, the greater part of the draft-chest could be embodied in the power-house substructure, and the length of the power-house would not have to be increased. If such a unit had been selected for the Lock 12 Development as described, considering the very large capacity of the units, it would probably have developed into a two-bearing independent turbine coupled to a two-bearing engine-type generator. It might have been advantageous to combine generator and turbine into a three-bearing unit with one continuous shaft, requiring a narrower power-house. In either case, although horizontal machines would have required a wider building, the foundations would have been very much simpler than they necessarily had to be for vertical units; and a comparison of costs of the two types of power-house building would be interesting to the profession. Doubtless such a comparison was made by the engineers of the power company before units of the vertical type were adopted.

One of the prime advantages of the horizontal-shaft unit is its greater accessibility and the greater ease with which the turbine may be dismantled. In the natural course of events, turbine runners of any type of unit are bound to wear, and must be replaced sooner or later. With a horizontal shaft and a scroll casing built up of a number of parts, the removal and replacement of a runner is much easier and much less costly, and takes a great deal less time than with a vertical-type unit where the entire stationary superstructure of the generator, the exciter, the thrust-bearing, the steady-bearing, the gate rigging, and the entire rotating part of the unit have to be dismantled before the turbine runner may be removed from the shaft and replaced by a new one.

Mr. Homberger. Highest efficiencies and economic construction may well be attained with volute or scroll casings of large horizontal shaft units, as recent practice both in the United States and in Europe has proved.

It does not seem impossible to the writer to design a horizontal, single-runner, single-discharge unit for the conditions as they are in the Lock 12 Development and to obtain satisfactory structural features throughout.

With reference to loss from the confluence of streams in the draft-tubes in multitudinal units, this does not seem to apply in the present case. If a single-runner, double-discharge unit had been chosen, these losses could have been avoided by arranging for two draft-tubes, and there would have been no confluence of streams. The question of installing units of the double-discharge and single-runner, or double-discharge double-runner type, had probably been examined to a considerable extent when the hydraulic equipment of the plant was under discussion. There could also have been selected a unit with two scroll casings, one for each runner, which would have promised a very high efficiency. This type of unit has become well known to the profession since the power-plant of the Ontario Power Company at Niagara Falls was started, about a decade ago. The prime advantage of this type of machinery would have been the possibility of selecting a turning speed about 50% higher than the one resorted to in the Lock 12 plant, resulting in the advantage of a less costly generator, and lighter parts to be handled, and indirectly in a saving in cost of installation, overhauling, and replacement. It would be interesting to learn what comparative investigations had taught in connection with the Lock 12 Development, prior to the adoption of the vertical units.

Another point in favor of the horizontal unit is the fact that the gate-operating mechanism, governor connections, etc., permit a much more convenient arrangement.

The question as to the highest efficiency which may be obtained with a scroll casing does not seem to be entirely settled yet; as a matter of fact, turbine runners tested at Holyoke in an open flume have sometimes shown higher and sometimes lower efficiency than they did finally in the plant when installed with the scroll casing for which the runners were originally designed.

Mr. Taylor. H. BIRCHARD TAYLOR,* ESQ. (by letter).—This very complete paper is a valuable contribution to the literature on modern hydro-electric plants. The progress made in the last few years in this field, particularly in respect to the turbines, has been very marked, but the literature on this subject has not kept pace with it; so that it is a real pleasure to read this paper. It not only describes one of the most modern and prominent developments in the United States, but

* Hydraulic Engr., I. P. Morris Co., Philadelphia, Pa.

also gives a considerable amount of valuable technical detail, which should prove of interest to engineers. Mr. Taylor.

The writer is particularly interested in this development, as the turbines were designed under his direction. The following comments will be confined to that subject.

The contract for the four 17 500-h.p. turbines was awarded in October, 1912, a short time after the single-runner, vertical-shaft, low-head turbine had come into use, and during the early stages of its development. At that time there had never been constructed a vertical-shaft, single-runner unit of so high a power to operate under a head as low as 68 ft., nor had there been used concrete volute turbine casings under a head as high as 68 ft. In two respects, therefore, the Alabama turbines represent a pioneer development.

Previous to the award of the Alabama contract, there had been a few plants completed or under construction in which the vertical-shaft, single-runner unit had been adopted, and in which the volute casings and draft-tubes were moulded in the concrete. Among these may be mentioned the two plants of the Appalachian Power Company, in Virginia; the plant of the Mississippi River Power Company, at Keokuk, Iowa; and the plant of the Georgia-Carolina Power Company, in Georgia. The turbines in these plants, however, were designed for lower heads and for much lower unit capacities. At this writing, the Alabama units are the most powerful vertical turbines of this type in existence, with one exception; namely, the 20 000-h.p. units of the Laurentide Company, Limited (head 76 ft.).

Table 7 gives a comparison of the powers, heads, and speeds in the plants just mentioned.

TABLE 7.—Comparison of Several Hydro-Electric Plants.

	Head, in feet.	Speed, in revolutions per minute.	Capacity, in horse-power.
Appalachian Power Company, Plant No. 2.........	49	116	6 000
Appalachian Power Company, Plant No. 4.........	34	97	3 500
Mississippi River Power Company..................	32	57.7	10 000
Georgia-Carolina Power Company..................	27	75	3 125
Alabama Power Company..........................	68	100	17 500

The Appalachian turbines were the first of this type to be put into operation, and the results were so satisfactory that the principal features of their design were adopted for the Alabama turbines, and have been used in a very large majority of recent installations.

The Appalachian turbines were tested after the award of the Alabama contract. The average of the maximum efficiencies secured in two units was found to be 93.7%, which, at that time, seemed almost

Mr. Taylor.

incredible, as this result was about 4% greater than the maximum efficiency secured in the model runner at Holyoke. This increase in efficiency over the model runner, as obtained in the vertical, single-runner setting, however, has been recently confirmed by the tests conducted in the plant of the Mississippi River Power Company, at Keokuk. The efficiency secured in the Appalachian turbines, along with the mechanical simplicity of the vertical-shaft, single-runner unit, has established that type for all important low and medium head installations.

The test of the Appalachian turbines should be of peculiar interest to the engineers of the Alabama Power Company, for the reason that the designs of the runners in the Alabama and Appalachian units were based on the same Holyoke model. As the settings of the turbines in both plants are almost identical, the efficiency of the Alabama units may be assumed to be at least as high as that secured in the Appalachian units, and probably slightly higher, in view of their greater size and higher power.

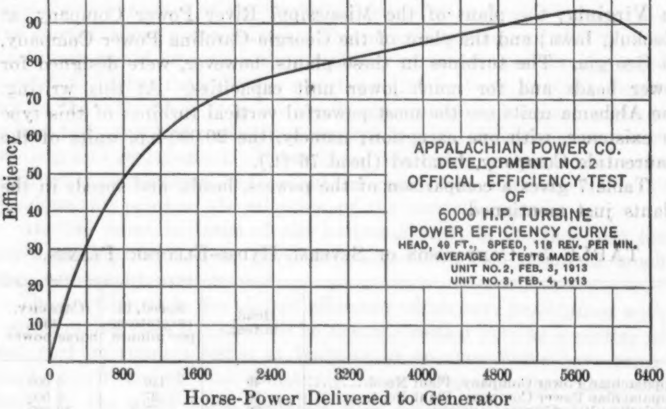


FIG. 55.

Fig. 55 shows the official efficiency curve of the Appalachian turbines. If we revise the horse-power scale, and substitute 17 500 h.p. for 6 000 h.p., the curve may be assumed to apply to the Alabama turbines. When the turbines are tested, it will be very interesting to learn just how closely the efficiencies obtained approach this curve.

During the past 2 years there have been constructed a great number of vertical-shaft, single-runner units for prominent developments —the I. P. Morris Company alone having constructed turbines of this type aggregating approximately 500 000 h.p.—but there have not

been any new features of importance developed since the design of the Alabama turbines. The lignum-vitæ guide-bearing; the speed ring; the direct-connected operating engines with trunk pistons; the location of the thrust-bearing above the generator; the use of the Kingsbury thrust-bearing; and the pneumatic brakes arranged to act against the lower face of the rotor, etc., have been pretty well accepted as standard details in subsequent work. Mr. Taylor.

The Alabama Power Company was one of the first to adopt the speed ring in connection with concrete volute casings. The design of the speed ring for these turbines was unusual, because of the high head involved. Aside from making the speed ring vanes of sufficient strength, when acting as columns, to carry the weight of the concrete floor above the casing, the weight of the pit liner, generator, and thrust loads down to the concrete below the casing, it was necessary to make them safe when subjected to tension. The rise in pressure in the volute casing during sudden closure of the turbine gates, when added to the already high normal pressure, would tend to lift the concrete floor above the casing, which must be resisted by tension in the speed ring vanes. The speed ring vanes, therefore, were made amply safe to withstand the greatest possible load in either direction which might be imposed.

The use of the speed ring, in connection with the Appalachian turbines, was the subject of considerable adverse criticism. The point was often raised as to whether the speed ring vanes, which are fixed, would not tend to peak the efficiency curve and reduce part load efficiencies. In other words, it was pointed out that these fixed vanes did not conform to the guide-vanes at all gate openings, but were suited to one gate opening only. The Appalachian curve, as shown in Fig. 55, shows no tendency to peak, and the part gate efficiencies cannot be said to be low. It should be remembered that at low gate openings, although the speed ring vanes do not come opposite the guide-vanes, as they do at or near the point of maximum efficiency, the velocity of the water is low, and a comparatively sudden change in the direction of flow when entering the guide-vanes is of no great moment.

It was also argued that the usual experimental outfit in which the Holyoke models are tested is not equipped with speed rings, and, as very high efficiencies have been secured at Holyoke, there is no reason for the use of speed rings in large turbines. Although it is true that very high efficiencies have been secured at Holyoke, the foregoing conclusions are not justified. At Holyoke we have an open flume, in which the velocities are comparatively low, though, in the case of large units with spiral cases, the usual velocities are quite high and represent very much higher percentages of the spouting velocity than those which exist in the Holyoke flume.

Mr. Taylor.

The principal advantages secured in the use of the speed ring are:

1st.—A speed ring insures a uniform direction of flow to all the guide-vanes. Although, as mentioned previously, this is not of importance at low gates, it is at the gate openings at and above those corresponding to maximum efficiency, where the velocities are high.

2d.—The projected area normal to the direction of flow of the speed ring vanes is materially less than that of tie-bolts. The lines of flow, therefore, are interrupted to a less extent by the speed ring vanes than by tie-bolts of circular section. The writer believes that when high velocities are encountered, the projected area normal to the direction of flow is about as important a consideration, as far as losses are concerned, as the actual shape of the tie-piece.

3d.—From an economical standpoint, speed rings are desirable, because without sacrifice in efficiency, they permit higher velocities in the turbine casing, therefore closer unit spacing and, consequently, a reduction in power-house dimensions.

4th.—The mechanical advantages derived from the use of the speed ring are sufficient for its adoption, aside from any hydraulic advantages. In erection, it is desirable to eliminate machine work and unnecessary fitting in the field. In the construction of the turbine in the shop, the speed ring is put on a boring mill, and, at one setting, is turned to suit the runner band, the lower distributor plate, the head cover, and the lower section of the pit liner, so that, when it is leveled up in the field, the location of all these connecting parts is definitely established.

The advantages which the speed ring offers in securing quick and accurate alignment of the entire turbine during erection, over any design involving separate foundation rings with independent tie-bolts, requiring the separate alignment of each ring, are obvious.

A comparison of the erection costs of two types of turbines, installed simultaneously by the same company, one with speed rings and one with tie-bolts, showed a very marked advantage in favor of the former type. As the units were otherwise similar, the saving in cost as well as the greater speed in erection was attributed to the advantages secured by the use of speed rings.

During the past 2 years the merits of the speed ring have been very generally recognized, and it has been specified by the engineers of a great number of prominent developments. It is interesting to note that the manufacturers who at first argued strongly against its use have recently provided it in connection with some of their contracts.

The authors' remarks, regarding turbine efficiency guaranties, as stated on page 2221; are not altogether clear. They state: Mr. Taylor.

"And the manufacturers have guaranteed that the units as installed will have a maximum efficiency of 87% when delivering to the shaft from 15 000 to 17 000 h.p."

This statement should read as follows:

"The manufacturers have guaranteed that the turbines will have a maximum efficiency of at least 87% when delivering a power between 15 000 and 17 000 h.p."

In other words, the turbines are not guaranteed to develop 87% efficiency for all powers between 15 000 and 17 000 h.p., but are guaranteed to develop a peak efficiency of 87%, such peak to occur between the limits of 15 000 and 17 000 h.p.

In making the preliminary studies of the Alabama units, the question of speed was given very careful consideration. The principal features studied in determining the speed were as follows:

1st.—The allowable elevation of the center line of the runner above low tail-water.

2d.—The corrosion of the vanes of high specific speed runners when operated under high heads.

3d.—Part-gate efficiencies.

4th.—Powers developed under heads below normal.

These questions will be considered in order.

1st.—Elevation of Runner Above Tail-Water.—With the development of high specific speed runners, the question of draft-tube design and the allowable elevation of the runner above tail-water has been given very careful attention. Until comparatively recently, it has been thought that the draft-tube was merely a device by which the turbine could be placed a certain distance above tail-water without losing the corresponding head, and it was generally thought that this distance was only limited by the height of the barometric column.

The draft-tube to-day, however, is looked on as one of the most important elements entering into the turbine, especially where high specific speed runners are involved. It is a function of the draft-tube, not only to enable the runner to be placed above tail-water level, but to regain the energy existing in the velocity of discharge from the runner vanes. The off-flow velocity head from high specific speed turbine runners may be as high as 50% of the total head acting on the turbine. The regaining of this velocity head by the draft-tube takes place in the form of a reduction in pressure or an increase in vacuum in the draft-tube, the maximum vacuum occurring immediately below the runner, where the discharge velocity is greatest.

Mr. Taylor.

With a draft-tube of 100% efficiency, the vacuum at the runner is equal to the static draft head or vacuum corresponding to the vertical distance to tail-water level, plus the head corresponding to the velocity of discharge from the runner. The sum of the static draft head plus the velocity draft head must not be greater, theoretically, than the barometric column (34 ft. at sea level). Actually, this sum should not approach this limit by less than 3 or 4 ft. The sum of the static head and velocity head in the draft-tube is, of course, reduced by the losses in the draft-tube and also by the head corresponding to the velocity at the discharge section of the draft-tube, but in modern plants these two elements may be disregarded, as they are kept within negligible values.

The margin between the allowable draft head and the theoretical maximum draft head, is provided principally for regulating purposes. Due to the inertia of the water column in the draft-tube, the sudden closing of the turbine gates increases momentarily the vacuum at the runner, and if this vacuum before closure is already up to the limit, the column of water may leave the runner and then return in a very strong upward surge. In some cases, this upward surge in the draft-tube has resulted in lifting the rotating element of the unit clear from the thrust-bearing.

If flood conditions make it necessary to place a turbine runner, say, 20 ft. above low tail-water level, the velocity draft head at the discharge of the runner at full load should not exceed, say, 10 or 11 ft. With an increase of specific speed, the velocity head through the band, or throat of the runner, must increase, because a runner, to develop a given power, decreases in diameter with an increase in specific speed. Therefore, a specific speed must be selected which will not give a greater velocity head from the runner than that equal to the difference between the 3 or 4 ft. less than the barometric column and the static draft head.

The static draft head of the units at Lock 12; that is, the vertical distance from the center line of the intake to the runner to low tail-water, is 20 ft. With an assumed efficiency, at 17 500 h.p. and 68 ft. head, of 90%, the discharge velocity head from the runner amounts to 10.4 ft. Under the maximum head of 74 ft., at the same gate opening, this velocity head would be increased to approximately 11.3 ft. The resulting maximum draft head possible is, therefore, 31.3 ft.

2d.—The Question of Corrosion as Affected by Specific Speed.—Unfortunately, there is no established law for determining the maximum specific speed allowable for any particular head, aside from that affecting the draft head, which has just been discussed.

As the specific speed of a runner increases, the width across the distributor or intake increases, and the diameter decreases. In addition to the velocity of discharge from the runner increasing with an

increase of specific speed, there are conditions set up, primarily at part gates, which make runners of this type very much subject to corrosion, caused to a great extent by the radical change in the direction of the flow lines for changes in gate opening. Mr. Taylor.

Aside from the question of draft head, it may be said that the safest method of determining the allowable specific speed for a given head is by consulting a curve drawn between specific speed and head, on which are plotted points corresponding to various installations, which have been in operation for a considerable time and are known to be successful in withstanding corrosion. This curve may be termed the "curve of experience".

In considering turbines for a new development, therefore, the revolutions per minute adopted should not call for a specific speed which would fall to any great extent above a curve thus drawn. It is advisable, of course, to reach out gradually from this curve, or, in other words, to extend our experience, but no great step should be taken at any one time.

At the time of the selection of the speed for the Alabama turbines, it was known that turbines in several installations had successfully withstood corrosion under conditions approximating those which would exist in the Alabama units when operating at a speed of 100 rev. per min.

The next two questions will be considered together:

3d.—The Question of Part Gate Efficiencies as Affected by the Revolutions per Minute of the Unit; and 4th.—Power at Low Heads.—One of the inherent characteristics of the performance of turbine runners is a decrease in the part load efficiencies with an increase in specific speed. There is, of course, a limit to this rule. In other words, by decreasing indefinitely the value of the specific speed and, consequently, the revolutions per minute of the unit, the part load efficiencies apparently cannot be raised above certain maximum values. As far as part load efficiencies are concerned, there seems to be an ideal specific speed by the adoption of which it is possible to obtain part gate efficiencies which cannot be improved on by further decrease in specific speed. The experience of the writer would indicate that this specific speed resulting in an ideal shape of efficiency curve, as far as part gate efficiencies are concerned, lies in the neighborhood of 120 (metric).

Fig. 56 will convey an idea of the manner in which these part gate efficiencies increase with decreased specific speed. The values of the specific speeds shown in this figure are in the neighborhood of that of the Alabama turbines (301.5). It will be noted that Curve *O* is of the highest specific speed, namely 370; that it develops a maximum efficiency of a trifle more than 90%; that the part gate efficiencies are lower than those of the remaining curves, and that

Mr. Taylor. the point of maximum efficiency occurs nearer full load than is the case with the remaining curves. Curve *M*, with a specific speed of 290, has a much higher part gate efficiency and also a slightly higher maximum efficiency, the point of this maximum efficiency occurring at a lower percentage of full load than on the remaining curves. Curves *A* and *R* represent specific speeds of 301 and 339, respectively. The shapes of these curves are consistent with those of *M* and *O*.

In applying the question of selection of speed directly to the Alabama turbines, it may be of interest to reproduce the performance curves which were discussed with the Alabama Power Company. These curves are shown in Fig. 57. It will be noticed that there are four efficiency-power curves, *M, E, A,* and *O*. These curves are stepped up from runners designated by these letters. The runners on which these curves are based, when stepped up to give the power indicated, would operate at speeds of 94.8, 100, 100, and 120 rev. per min. in the case of types *M, E, A,* and *O* runners, respectively.

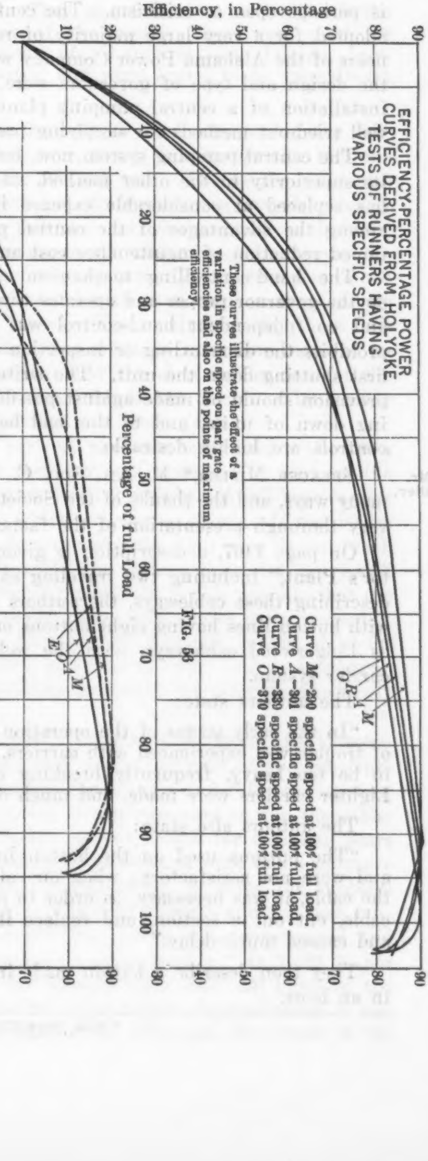
It was decided that though the selection of Type *M* runner would result in extremely high part gate efficiencies, the fact that this runner would not give high power at heads below normal and that the revolutions per minute resulting would necessitate a comparatively expensive type of generator, caused its elimination.

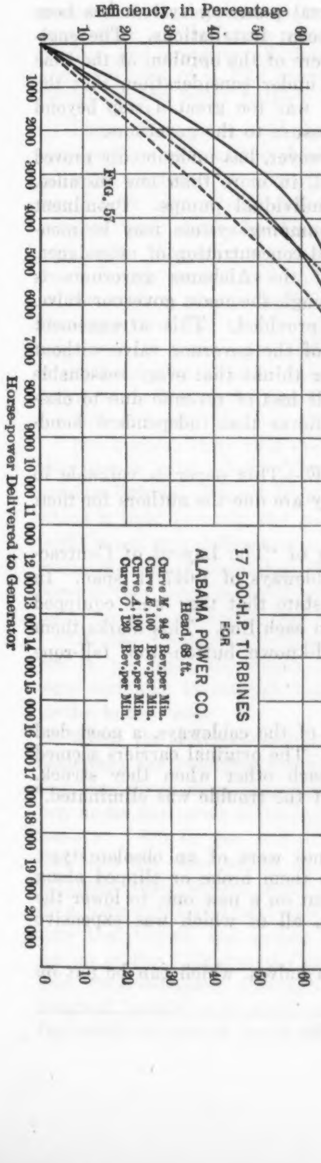
The speed possible with a runner similar to Type *O*, namely 120 rev. per min., was very desirable, but the part gate efficiencies secured were so low, when compared with the others, as to make its adoption unwarranted. Also, the selection of a runner having so high a specific speed would possibly have rendered the runners susceptible to corrosion, as discussed previously (being too great an advance beyond the "curve of experience"); also, it would have been necessary to place this runner at an elevation 5 ft. below that fixed for the runner actually used, and this was found to be objectionable.

The comparative virtues of Runners *A* and *E* were next considered. As both these runners were capable of delivering the same power at the same revolutions per minute, the question of speed, of course, did not figure. The advantage of higher part gate efficiencies of Runner *E* was, it was thought, partly offset by the fact that this runner developed under the lower heads less power than Runner *A*. Further, the dimensions of Runner *E*, when stepped up to the Alabama conditions, resulted in a considerably higher width of distributor, or height of guide-vane. Also, Runner *E*, when enlarged to suit the Alabama conditions, would be materially heavier and, therefore, more costly than Type *A*. The fact that the latter runner made possible a very low width of distributor and a runner of very inexpensive design, and also the fact that it developed under the low head conditions comparatively higher power, resulted in its selection.

One or two thoughts have suggested themselves to the writer in regard to the governors.

Mr. Taylor.


EFFICIENCY-PERCENTAGE POWER CURVES DERIVED FROM HOLYOKE TESTS OF RUNNERS HAVING VARIOUS SPECIFIC SPEEDS
These curves illustrate the effect of variation in specific speed on part gate efficiencies and also on the points of maximum efficiency
Curve M—290 specific speed at 100% full load.
Curve A—301 specific speed at 100% full load.
Curve R—339 specific speed at 100% full load.
Curve O—370 specific speed at 100% full load.
Efficiency, in Percentage
Percentage of Full Load
FIG. 58


17 500 H.P. TURBINES FOR ALABAMA POWER CO.
Head, 68 ft.
Curve M, 93.8 Rev. per Min.
Curve E, 100 Rev. per Min.
Curve A, 100 Rev. per Min.
Curve O, 120 Rev. per Min.
Efficiency, in Percentage
Horse-power Delivered to Generator
FIG. 57

Mr. Taylor. The adoption of individual pumps for supplying pressure to the governors in a plant containing as many as four units (six eventually), is perhaps open to criticism. The central pumping system has been adopted for a very large majority of recent installations. The engineers of the Alabama Power Company were of the opinion, at the time the design and type of governors were under consideration, that the installation of a central pumping plant was too great a step beyond well tried-out methods for supplying pressure to the governors.

The central pumping system now, however, has undoubtedly proved its superiority to the other method, and, in more than one instance, has replaced at considerable expense individual pumps. Prominent among the advantages of the central pumping system may be mentioned reduction of maintenance cost and concentration of attendance.

The hand-controlling mechanism of the Alabama governors is on the governor proper and operates through the main governor valve, and no independent hand-control was provided. This arrangement prohibits the dismantling or inspection of the governor valve without first shutting down the unit. The writer thinks that every reasonable provision should be made against possible loss of revenue due to closing down of units, and to this end believes that independent hand-controls are highly desirable.

Mr. Miller. SPENCER MILLER,* M. AM. SOC. C. E.—This paper is valuable in many ways, and the thanks of the Society are due the authors for their very thorough presentation of the facts.

On page 1467, a description is given of "The Layout of Contractor's Plant," including two traveling cableways of 1 647 ft. span. In describing these cableways, the authors state that they were equipped with button lines having eight buttons on each line. This marks them as Lidgerwood cableways, with the well-known button-stop, fall-rope carrier system.

The authors state:

"In the early stages of the operation of the cableways, a good deal of trouble was experienced with carriers. The original carriers seemed to be too heavy, frequently breaking each other when they struck. Lighter carriers were made, and much of the trouble was eliminated."

The authors also state:

"The buttons used on the button lines were of an obsolete type, and were not satisfactory; when one of them broke or slipped along the cable, it was necessary, in order to put on a new one, to lower the cable, cut out a section, and replace it, all of which was expensive and caused much delay."

They then describe a button made in halves, which can be put on in an hour.

* New York City.

Again, the authors state: Mr. Miller.

"The cableways handled practically 60% of the concrete, and were run both day and night. They also handled practically all the excavation in the early stages of the work. They were used for moving and erecting all derricks, etc., handling form lumber, iron, and materials of a like character, and were found to be exceedingly useful for the latter purpose."

The authors have said that the buttons used for spacing the fall-rope carriers were of the obsolete type, and it is probably true that the fall-rope carriers were also of the obsolete type. The results, no doubt, would have been wholly different had the modern high-speed cableway carriage and shock-absorbing fall-rope carriers, as well as buttons, been used. As indicated in the paper, the efficiency of a cableway depends largely on the fall-rope carrier system. The system mentioned by the authors was probably identical with that used on the Chicago Drainage Canal on twenty cableways. The repair costs for these fall-rope carriers were considerable. The constant attention required in strengthening the button fastenings and the slots in the head of the fall-rope carriers showed an inherent fault in the design. The remedy for this difficulty was not discovered for many years.

The authors state that the carriers were too heavy and broke when they struck each other. This they say was remedied by making the carriers lighter. Impact is in proportion to weight. A fall-rope carrier which is light is weak, and one which is heavy is strong. If the design is the same, it is difficult to understand why the light ones do not break when the heavy ones do. Probably the lighter ones did not break because they were made of better material. It is also possible that the light carriers worked better because they were better proportioned.

On the Chicago Drainage Canal, with carriage speeds of about 800 ft. per min., the repair costs and loss of time, due to breakages, were serious. It seemed impossible to secure the buttons effectively to the button rope.

When the Lidgerwood Company undertook to build the thirteen cableways for handling concrete at Panama, no part of the construction was given more attention than the buttons and fall-rope carriers, for, under the terms of the contract, it was necessary to run the cableways at 1 850 ft. per min. These carriers, which for so many years had been the chief source of delays on cableways, were taken up more seriously than before, with the determination to solve the problem of a high-speed fall-rope carrier. The best carriers and the best buttons that brains and money could then devise were constructed and set up at the Lidgerwood Company's testing station. At 1 100 ft. per min. they collapsed, although the buttons held fast. All kinds of the toughest metals were used, and yet after weeks of experiment it was impossible to obtain anything that would withstand the shock of the

Mr. Miller. button at a speed in excess of 1 150 ft. per min. Finally, after weeks of trial and continual rebuilding, the shock-absorbing principle was invented. The head of the fall-rope carrier was hinged instead of being rigidly connected to the body of the carrier. The button collided with a swinging eye instead of an elongated slot, and, instead of colliding at two points on opposite sides of the button, collided almost throughout its entire diameter.

As soon as this construction had been set in motion, a speed of 3 000 ft. per min. was developed, and the old law, that the impact was in proportion to the weight and increased as the square of the velocity, was emphasized in an unusual manner.

What the authors' conclusions would have been had the new shock-absorbing carrier and carriers been used is not difficult to anticipate.

The cableway engines used did not embody the numerous improvements now in use by the manufacturers of those engines, such as improved cylinder design and construction, larger, heavier, and better balanced reciprocating parts, etc., all resulting in higher speed and efficiency.

The obsolete buttons which the authors refer to were secured by a through pin and bearing in solder or Babbitt. The new buttons are taper-bored, the strands of the rope being opened and a tapered pin inserted. The taper of the bore and the pin is the same, and is less than the angle of repose. No other pin is used. The more the carriers hammer the buttons, the tighter the fastening. At Panama heavy jacks were necessary to remove the buttons after they had been in service for several months. The new buttons do not slip, and do not injure the button rope. Button ropes now frequently last four and five years. Therefore, the necessity for a divided button is not apparent.

The official reports at Panama show that these cableways at Gatun were kept at an efficiency of 99 per cent. Expressed in other terms, the entire cableway was ready for operation 99 min. out of every 100 they were called on for service, during the period recorded, and that applies not only to the fall-rope carriers, but to every part of the cableway. The efficiency of the new fall-rope carriers, therefore, can confidently be stated to be even in excess of 99 per cent.

On page 1495, the authors state:

"A cableway is not satisfactory in handling excavated materials, as it is not only expensive, but slow."

They also state:

"The writers are of the opinion that, as a rule, the cableway should be restricted to moving plant, derricks, forms, and general work, and also such concreting as is advantageous to place with it, and that it

should not be made the main method for the transportation of all materials on the job, as it was here." Mr. Miller.

Among those who use cableways extensively for concreting operations, it is a common expression that the cableway is worth its investment merely for the purposes of moving plant, forms, etc., but the experiences of the Isthmian Canal Commission with cableways at Gatun Locks, Panama, by the John F. Casey Company, at Aspinwall, Pa., and Cleveland, Ohio, and by the H. E. Talbott Company, Grand Mére, Que., Canada, hardly bear out the conclusions of the authors.

The authors state that the record for a 10-hour run with the cableways was seldom more than 400 cu. yd. Does this mean 400 yd. for two cableways? At Panama runs as high as 60 cu. yd. per hour, with one cableway handling 2-yd. batches, were recorded. The John F. Casey Company, on thin section baffle walls, 6 ft. 9 in. wide at the bottom and 2 ft. wide at the top, has laid as much as 692 cu. yd. per day with one cableway. The H. E. Talbott Company placed, with one cableway, from 320 to 350 cu. yd. of concrete, with a travel of 1 100 ft., in 10 hours. Some extraordinary records are being made at Cleveland, Ohio, at present, by the John F. Casey Company. No other device will do what the cableway is adapted for, namely, handling excavation, placing concrete and large rock, moving forms, timber, derricks, and miscellaneous material, such as ironwork, etc. If time was limited, there is no reason why three or four cableways should not have been used.

In dam construction, also, there is nearly always the menace of high water during a part or all of the construction period. No device presents such effective insurance against that danger as the cableway. It stands safe throughout, and for removing auxiliary plant and material quickly when occasion demands, it is always ready. The speaker believes that had the latest type of high-speed cableways been used on this work, the authors would have modified their deductions.

W. E. MITCHELL,* ESQ. (by letter).—Referring to the discussion by Mr. R. D. Coombs, on this subject, Mr. Coombs, of course, did not know, what the authors of this paper knew, namely, that it is the intention to have a paper, discussing the purely electrical side of the development in Alabama, written by the Electrical Engineer for the American Institute of Electrical Engineers. When this paper comes out it will answer a number of the points raised by Mr. Coombs. Mr. Mitchell.

It may be stated here, however, that probably more attention than usual was given to the subject, not only of the transmission line voltage, but of the style and size of lines for both transmission and distribution. These matters were gone into very thoroughly, following

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Mr. Mitchell. a careful survey of all high-tension systems in operation at the time, with the definite idea of making the service as reliable as possible.

Following out this idea, Birmingham, which of course is the main distributing point, is supplied by three 110 000-volt circuits, two by one route going practically direct to the city limits, and the third by an entirely separate route, so that the likelihood of interruptions due to severe storms in one locality will be minimized.

As stated in the paper, the transmission-line voltage is 110 000 volts, and the distribution lines are practically all of 22 000 volts. The middle cross-arm should have been given as being 5 ft. longer than the top and bottom cross-arms, making the offset of the middle wire 2 ft. 6 in.

The writer cannot agree with Mr. Coombs' statement that the high towers will not permit of the longer spans in rough country. There are times, of course, when considerably shorter spans are made necessary by intervening hills, but in general it was found in Alabama that with Type A towers the average span was 700 ft., and that in very few cases did it drop below this, although it was frequently as much as 750 or 800 ft.

The Type C towers, which, as a matter of fact, were used in the flat country, were not purposely designed for this work, but with the idea that steel-core aluminum would be used on their lines, as this called for less sag and for anchor towers at least every tenth tower. After the towers were ordered, however, it was decided to use copper, and hence the spacing had to be reduced. In general, a higher tower would be more economical in flat country. The Type C tower was designed light on purpose, as it would have been impossible to have anchored the steel-core aluminum at every tower, and it was the idea to anchor it at every tenth tower on an extra heavy Type B tower.

In regard to the specifications covering clearance and loading, a very careful investigation of the climatic conditions in Alabama was made, and it was found that only about three times in the last 20 years had conditions been such that sleet would have been formed; therefore it is thought that the use of ¼ in. of ice and 8 lb. of wind pressure was sufficiently conservative. The standard tower was used on all spans, as it was found to be still well within safe limits. Excessively long spans were purposely avoided in order that there would be no special construction, always extremely expensive. The only special towers used on the whole work were four for two river crossings, and the dead-end towers at each sub-station.

Referring to the criticism of distribution lines built on wooden poles, this also received careful consideration, and investigation of the life of creosoted poles throughout the South was made. Long-leaf yellow pine is extremely plentiful in this district, and it was found

that poles treated with 12 lb. of creosote showed no deterioration after 20 years. These poles proved to be very much cheaper than steel, although, for comparative purposes, the company has actually put in one 25-mile line with flexible towers. This ran up the cost, however, to about 15% more than the same line built on creosoted poles. The writer does not believe that the steel poles would have enough longer life than the creosoted poles to justify them. Mr. Mitchell.

The size of the conductors on the 22 000-volt lines varies from No. 4 to No. 00 copper. The standard spacing, using creosoted, long-leaf, yellow pine cross-arms, is 5 ft. horizontally and 3 ft. 6 in. vertically between conductors. This is believed to be a more liberal spacing than is ordinarily used.

The writer was on this development from the commencement of the work, and has had the opportunity to be in direct charge of the operating from the time it went into commission up to the present date. During the past lightning season there was considerable trouble from punctured insulators, and one more disk is now being added to all strings. The entire system is now being inspected, and all insulators are being tested with a megger. It is believed that this will eliminate much of the trouble in the coming lightning season. On the distribution lines, where a 35 000-volt pin insulator has been used, there has been practically no trouble whatever.

The generating plant has worked out fully up to the expectations, with regard to simplicity of design, and has proved a good station to handle, from the operating point of view.

GARDNER S. WILLIAMS,* M. AM. SOC. C. E.—The illustration of the draft-tube showing the cradle (Fig. 39), takes the speaker back to the construction which it fell to his lot to design and supervise at Sault Ste. Marie, in 1906, for the Edison Sault Electric Company. There the so-called cradle was used for setting the draft-tube, and one of the first curved concrete draft-tubes in the United States was built. A method utilized there—by the same contractors who constructed the plant described by the authors—was that, instead of wooden lagging on the form, hard wire cloth of ½-in. mesh was used, which would conform itself readily to warped surfaces, the main ribs being placed in the same manner as indicated by the authors. This hard wire cloth was tacked over the ribs, and outside of it canvas was stretched. This construction proved to be appreciably cheaper than wooden lagging, and secured very nearly as good results. Of course, the concrete surface was not quite as smooth as that against the smoothly planed wooden form would have been, but it was smooth enough for the purposes required. Mr. Williams.

* Ann Arbor, Mich.

Mr. Williams.

In that same plant, there was introduced probably the first concrete scroll wheel-pit used in connection with a water-wheel installation of low head, at least in the United States.

Considering draft-tubes brings up the point that there is considerable importance in their form. What may seem like a comparatively insignificant change in form may have an appreciable influence on the performance of the wheel. In relation to this, the speaker had an interesting experience within the past 6 months. About a year ago, a contract was made with a prominent wheel maker for a pair of wheels to be mounted in identical pits, and to be of identical capacity and characteristics. The contract called for a forfeit if the performance fell below the maker's guaranty of efficiency, which was 3% below the Holyoke rating of the model wheel, and a bonus if it were above the Holyoke test. When the plans of the draft-tubes were submitted to the maker, he declined to guarantee the performance of his wheels with the draft-tubes as designed, and sent a drawing of what he would accept. The speaker had some rather serious objections to adopting the type submitted, but it was arranged that one of the tubes should be built according to that plan and one according to the original design, and that the performance of the wheel would be determined on the tube built according to the maker's design. Dynamometer tests were made, the water being measured over a weir, and the draft-tube according to the original design gave an efficiency 3% higher than that of the other. The maker suspected that there was something in the pull of the dynamometer that might cause an interference with the action of the wheel, and, when the generators were installed, another test was made which gave similar results. Vacuum gauges were then attached at the wheel outlets, and it was found at once that the tube constructed according to the original design was showing a materially higher vacuum than the other. The final result gave about 7% more power and 5% better efficiency for the tube designed according to the speaker's theory.

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E. L. Sayers* and A. C. Polk,† Members, Am. Soc. C. E. (by letter).—The writers are pleased at the extent of the individual discussions on this paper, and will be glad to answer to the best of their ability the many questions brought up. Their only regret is that there was not a full discussion on the more general topic of modern methods of power development.

Shortly after the paper was published, one of the writers received a letter from Clemens Herschel, M. Am. Soc. C. E., asking a number of questions which he suggested could be answered in the closing discussion.

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† Parkersburg, W. Va.

Mr. Herschel wished to know: Messrs. Sayers and Polk.

1st.—What reduction of head would there be in times of flood, and why were not fall increasers used?

2d.—What was the method of determining the size of the installation?

3d.—He thought that too much importance has hitherto been assigned to the "nature and extent of the power market" and to its consequent load factor. He thought that load factors are transient things, with a consequent tendency to disappear, or to become equal to one, in course of time, and the progress that time brings with it; but that power-plants, and hydraulic works generally, are built to last as long as any known handiwork of man.

Taking up Mr. Herschel's first question: The maximum back-water effect which may be expected annually is 6 ft., reducing the maximum head to 62 ft., at which head the output of the wheels will be 16 000 h.p. As noted in the paper, there will be below Lock 12 another dam at the site of Lock 14, on which there will undoubtedly be flood-gates for controlling the level of the lake formed by it. The maximum available head now is 72 ft., which will be reduced to 68 ft. by the construction of this lower dam.

Under the greatest flood condition known to have existed, the head would be reduced to 54 ft., at which head the output of each wheel would be reduced to 13 500 h.p.

The matter of back-water was studied carefully from every angle, and the figures were checked by the establishment of gauging stations on the river and by the testimony of the oldest inhabitants.

Fig. 58 shows the rating curve constructed as the result of a great many readings, made first from boats and later from an overhead cableway stretched across the river at a point about ½ mile below the dam. The river at this point is considerably narrower than at the site of the dam, and on this account the fluctuations of level there are more pronounced than in the tail-race. At first, the readings were made with the company's meter alone, but, later, they were taken with both this meter and one belonging to the U. S. Geological Survey which, immediately prior thereto, had been calibrated at Washington.

With reference to fall increasers, it should be noted that each of these units uses 2 600 sec-ft. of water under full head, and fall increasers, though no doubt practical on relatively small installations, would not be practical in the case of such large units. Another vital objection to their use is that they involve increased investment and the establishment of large gates under high heads, with the consequent leakage, while the limiting factor in most hydro-electric developments is the careful conservation of the low-water flow. Another consideration involved is the fact that for periods of low water it was necessary

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to provide a steam auxiliary plant as a standby. This being the case, the construction of fall increasers would be an economical error.

With regard to the second question, relating to the size, it may be stated that the installation finally chosen was selected because one which at 80% power factor would produce 45 000 kw., under a safe assumption as a load factor, was calculated to be profitable. The wheels will carry this load under a head of 56 ft., and when the head is less it will be necessary to use the steam plant.

Using the conditions of 1897 for purposes of computation, a regulated flow of 2 650 sec-ft. can be obtained, using a draw-down of 10 ft. on the pond level. At the present head, this equals 286 000 kw-hr. per 24 hours. The Gadsden steam plant, at 94% power factor, will produce 225 000 kw-hr. per 24 hours.

The primary power available for sale under these conditions, therefore, would be 511 000 kw-hr. per 24 hours, and, at 50% load factor, this would be equivalent to an installation of 43 000 kw., maximum demand.

As a matter of fact, the actual load factor is greater than 60%, although 50% was assumed, which means that the plant will have a certain amount of secondary power to sell, or that the steam auxiliary must be extended when it becomes desirable.

The ultimate installation of six units will produce, in 24 hours, 933 000 kw-hr., estimated as follows:

Units.		Kilowatt-amperes.		Power factor.		Load factor.		Hours.		Kilowatt-hour.
6	×	13 500	×	80%	×	60%	×	24	=	933 000

This is available from the river 66% of the time, and it will require a steam auxiliary of approximately 30 000 kw., determined as follows:

Total output in 24 hours...........................	933 000 kw-hr.
Lock 12 production, dry period....................	286 000 " "
	647 000 kw-hr.

$$647\,000 \div 24 = 27\,000 \text{ kw., say, } 30\,000 \text{ kw.}$$

It is to be remembered that the locality in which this power is marketed has a very low-priced coal.

The writers do not agree with the apparent contention of Mr. Herschel that load factors are to be disregarded, or assumed as equal to unity. In certain communities it would be impossible to attain this condition. In a New England district of factories working 8 or 10 hours per day, and where the lighting and night street-car loads would be small, it is entirely reasonable to assume that the load factor would remain small for all time; but in a community where electric furnaces, cement mills, electro-chemical works, etc., abound,

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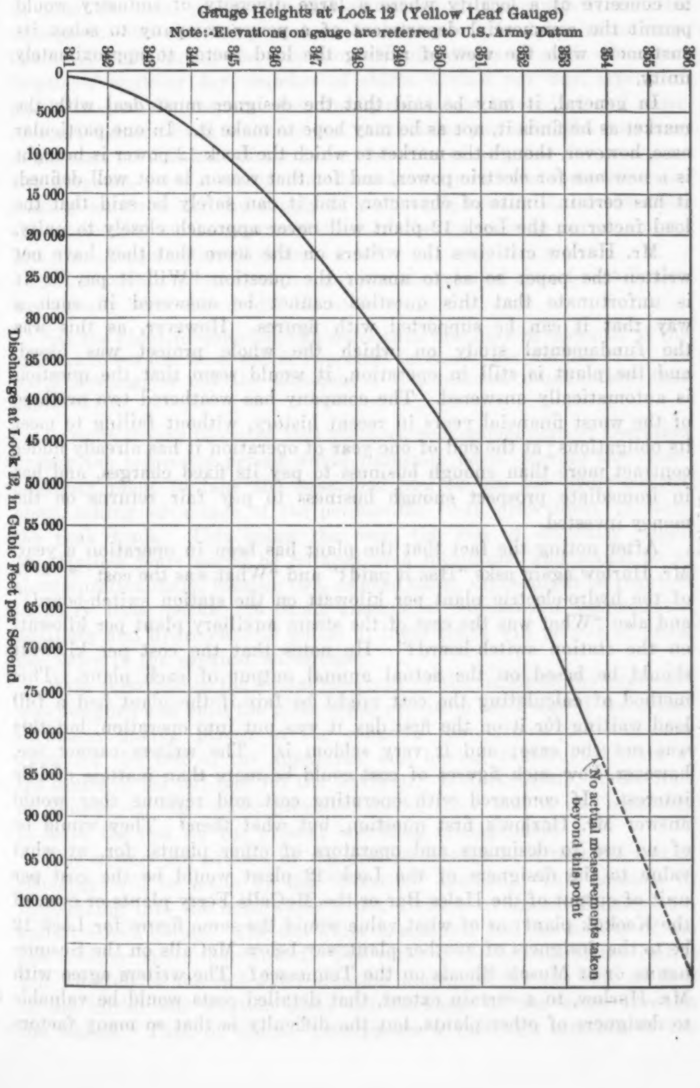

FIG. 58.

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the load factor would be large, and after the load was once taken by the plant it would remain about stationary. Of course, it is possible to conceive of a locality where a large diversity of industry would permit the commercial department of a power company to select its customers with the view of raising the load factor to approximately unity.

In general, it may be said that the designer must deal with the market as he finds it, not as he may hope to make it. In one particular case, however, though the market to which the Lock 12 power is brought is a new one for electric power, and for that reason is not well defined, it has certain limits of character, and it can safely be said that the load factor on the Lock 12 plant will never approach closely to unity.

Mr. Harlow criticizes the writers on the score that they have not written the paper so as to answer the question "Will it pay?" It is unfortunate that this question cannot be answered in such a way that it can be supported with figures. However, as this was the fundamental study on which the whole project was based, and the plant is still in operation, it would seem that the question is automatically answered. The company has weathered two or three of the worst financial years in recent history, without failing to meet its obligations; at the end of one year of operation it has already under contract more than enough business to pay its fixed charges, and has in immediate prospect enough business to pay fair returns on the money invested.

After noting the fact that the plant has been in operation a year, Mr. Harlow again asks "Has it paid?" and "What was the cost * * * of the hydro-electric plant per kilowatt on the station switch-board?" and also "What was the cost of the steam auxiliary plant per kilowatt on the station switch-board?" He notes that the cost per kilowatt should be based on the actual annual output of each plant. This method of calculating the cost might be fair if the plant had a full load waiting for it on the first day it was put into operation, but this was not the case; and it very seldom is. The writers cannot see, however, how such figures of cost could be more than matters of idle interest. If compared with operating cost and revenue they would answer Mr. Harlow's first question, but what then? They would be of no use to designers and operators of other plants, for, at what value to the designers of the Lock 12 plant would be the cost per unit of output of the Hales Bar or the McCalls Ferry plants or even of the Keokuk plant; or of what value would the same figure for Lock 12 be to the designers of another plant, say below McCalls on the Susquehanna or at Muscle Shoals on the Tennessee? The writers agree with Mr. Harlow, to a certain extent, that detailed costs would be valuable to designers of other plants, but the difficulty is that so many factors

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enter into costs that, unless one is acquainted with all the details of the construction, the figures are practically valueless. If one had in detail the costs of the material entering into the concrete, and the reasons for the cost being such, and the cost of the labor of mixing, handling, and placing the concrete, with details as to rates of wages, length of working day, number of shifts worked per day, layout of plant, flood conditions, etc., etc., the figures might be studied and prove of some value. Even the cost per rated horse-power of the turbines, or the cost per kilowatt of the generators, are of no value for comparative purposes, for the reason that there is at times keen competition in seeking contracts for the supply of such equipment, and, besides this, the personality of the representatives of the contracting parties enters into the case in no small degree.

Mr. Harlow's question with regard to the selected size of installation has already been answered. He asks:

"At the plant in question, can more than one unit be operated at the extreme low-water flow, and if two must be run to get the use of all the water flowing, is not the efficiency being reduced much below that expected?"

In reply it may be stated that the most economical plan of operation during the low-water period would naturally be to operate the steam auxiliary plant to take the base load and to use the hydro-electric plant, taking advantage of the permissible 10-ft. draw-down, to carry the peak loads. This being the case, the loss of efficiency which Mr. Harlow fears would be reduced to a minimum, and would be of short duration daily, because two, or even three, units might be required to carry these peaks. A small amount of adjustment between the auxiliary and hydro-electric plants at these times will permit of using the water to the greatest advantage.

With reference to the actual efficiency of the turbines, the engineers who designed and built the plant made provision for a test of the actual efficiency, having placed in the concrete foundations the necessary ducts for the test of the apparatus, and having constructed piers at the limits of the tail-race for the test weir. Unfortunately, at the time of completion, high water prevailed in the river, and it was necessary to leave the test for the present operating management.

The writers are advised by the present management that the test of efficiency will be made by the "titration" method. This consists in introducing into the head-water a salt solution of known density at a known constant rate. The tail-water is sampled and the density of the salt solution again determined. The ratio of the density of the initial salt solution to the tail-water solution multiplied by the rate of flow of the initial salt solution gives the discharge of the water through the wheels. This, of course, presupposes a thorough mixture of the

Messrs. Sayers and Polk. salt solution with the water flowing through the wheels. The details are matters of careful mechanical and chemical adjustment. Remarkable accuracy is claimed for the method.

The art of turbine building has come to such a point in recent years that it is not quite as unsafe as one might infer from Mr. Harlow's remarks to "depend on the Holyoke tests of a (comparatively) small turbine and then the guess of the builder that the full-sized turbines will 'fill the bill'." The guess of the builders has been so successful in the design of some recent turbines that the guaranteed efficiency has not only been exceeded, but the actual efficiency has exceeded by wide margins the Holyoke tests of the model runner. The Appalachian Power Company's units, which are of the same model as the Lock 12 units, attained an actual efficiency in test 4% greater than the maximum efficiency secured in the Holyoke test. This is a matter of unprejudiced record.

"The hydro-generator is now going toward the vertical; shall we, after a while, swing back to the horizontal?" Probably not. The tendency is toward large units to save cost per horse-power installed, and large units and horizontal shafts do not go well together. First, because it is possible to obtain higher efficiencies in large units with vertical shafts; second, because in most cases it is possible to design a much smaller power-house; and third, because the design of a satisfactory horizontal shaft for such large units as those at Lock 12 is practically impossible, due to the very heavy flexural stresses.

Mr. Coombs complains in his discussion as follows: "Unfortunately, the authors have followed the usual practice and dismissed the subject of line construction in a very sketchy way." The complaint is probably justified in a way, but the avowed purpose of the paper was to describe the design and construction of the Lock 12 dam and power plant. The same complaint might be urged against the manner in which the construction of the Gadsden steam plant, the sub-stations, and the study work on proposed developments were treated in the paper. There were very interesting features in all this work, but a treatment of them all in detail would have lengthened the paper beyond all reason. Mr. Mitchell, the present Operating Manager of the company, who was the Electrical Engineer in the construction organization, has answered the points brought up by Mr. Coombs.

Replying to Mr. Cooper, the thrust-bearings, as located at the top of the generator frame, seem to the writers to be satisfactory, and thus far no trouble has been traced to this arrangement. In fact, one of the writers has just completed the installation of three of these bearings on 10 000-h.p. machines, and no trouble is expected from them. There was some trouble at Lock 12, due (as stated to the writers) to foreign matter

getting into the oiling system. The writers agree with Mr. Cooper in his appreciation of the Kingsbury type of bearings, and only disagree with him in the extent of their praise. Mr. Cooper believes that these are the coming bearings for vertical shafts under heavy loads, but the writers believe that they were coming some 4 years ago, and that they have now arrived. Answering Mr. Cooper's question, no machine work of any kind was done on the castings in the field, and none was necessary. There was no trouble whatever from the castings being thrown out of line by the concrete. A speed ring was first set exactly and grouted in firmly, then all other castings, which had been previously fitted to the speed ring at the factory, were placed and the whole was grouted, the concreting operations taking 2 days. Messrs. Sayers and Polk.

Mr. Homberger says "With first-class design, vertical-shaft turbines of this particular type, of such huge dimensions, could hardly be otherwise [than self-contained]; each, of necessity, must be a homogeneous structure, or it would be impossible to make a satisfactory job of the installation in the plant." This statement is decidedly in error. The Keokuk units, which structurally are larger than the Lock 12 units, were built with incomplete pit liners. At Keokuk these were made up of rolled steel, and they required an immense amount of expensive fitting in the field. In fact, all units (or practically all) that were built prior to the construction of those at Lock 12 were constructed with incomplete pit liners. Mr. Taylor deals fully with this feature of the design, and it is hardly necessary to add to his remarks.

The taper fit of the runner on the shaft, referred to by Mr. Homberger, was selected primarily because it permitted the easy removal of the runner by the use of screws. This facility of removal may be worth considerable money in an emergency, and in reality costs very little more than a well-executed parallel press fit.

Mr. Homberger's questions with regard to the selection of the runner for the turbine and the location of the point of maximum efficiency are well answered in Mr. Taylor's discussion and by the curves of efficiency which he has shown.

The reason for the selection of vertical rather than horizontal shafts has been dealt with in the paper, and has been again referred to herein. It would be impossible, without devoting great space to the subject, to give the detailed studies which were made prior to the selection of the vertical-shaft units.

The writers fail to see the "prime advantage" of the horizontal-shaft unit in so far as greater accessibility is concerned. It is a question as to whether or not a scroll casing built of a number of parts, always remembering that these are units of enormous capacity, can be dismantled more readily than a vertical unit, with the assistance of a 100-ton crane. Furthermore, if one is to believe Mr. Homberger's

Messrs. Sayers and Polk. statement in the paragraph criticizing the taper fit of the runner on the shaft, where he implies that the runner will seldom be removed, this supposed "prime advantage" is not of much importance.

Mr. Taylor has contributed a very thorough discussion on the modern design of turbines and on various correlated questions, and the writers feel sincerely indebted to him for thus supplementing the paper. His criticism of the fact that no independent hand-control of governor was provided is slightly in error; it is not necessary to shut down a wheel to repair the governor. The mechanism is arranged so that the dash-pot can be cut out and hand operation used. The pilot-valve, it is true, cannot be cut out, but there is little about it to get out of order, and even then the addition of two ordinary stop-valves will allow the removal of the pilot-valve. This addition, however, seems to be of questionable advantage because the careless closing of one of these valves would permit a runaway. When running on hand-control, with these cut-outs made as outlined above, the gates are set at a constant opening, and a second turbine running on the governor control will take the variation in load. After nearly a year's operation it has not been found necessary to take out the pilot-valve on any unit for any purpose whatever.

Replying to Mr. Miller's remarks, the writers offer the following: There is no doubt that great improvements have been made in the type of buttons and fall-rope carriers used at Lock 12, there being considerable room for improvement in this design and the writers can well understand that the type of carrier and button used at Lock 12 could not be used at a speed of 1 850 ft. per min. The new carriers built to replace the old ones followed the general design of the old ones, but, as stated in the paper, they were of a lighter material, and were proportioned a little differently. Unfortunately, no detailed record was kept of the changes made, the carrier finally used being the result of several small changes in the general make up. Since the completion of the Lock 12 work, the writers have seen carriers built on the shock-absorbing principle, and consider them a great improvement. In the 18 months of service at Lock 12 each cableway wore out two button lines, so that the improvement along this line is indicated by Mr. Miller when he states that button lines now frequently last 4 or 5 years, this being a longer period than has ever come under the writers' observation. The maximum run of the cableways at Lock 12 was about 450 cu. yd. of concrete per cableway, per 10-hour day, the average travel being about 600 ft. The writers cannot agree with Mr. Miller that no other device will handle excavating, large rock, and concrete as well as a cableway. Mr. Miller has probably never stood under a long cableway and attempted to place a large stone swinging up and down on the end of the fall line, or attempted to land a skip loaded

with excavation on a dumping platform for loading into a car where the materials must be disposed to a dump. It is an exceedingly slow method, much time being lost in spotting, and the position for the man on the platform unhooking is precarious. The writers do not agree that it is the best method for handling concrete, either, their experience being that a dinky and cars to a derrick are about 25% faster than cableway to derrick. For other general work around dam construction, it is certainly a most handy tool, and one that should always be given its place in the plant. Messrs. Sayers and Polk.

Messrs. Sayers and Fritz.

with excavation on a dumping platform for loading into a car where the materials must be disposed in a dump. It is an exceedingly slow method, much time being lost in spotting and the position for the man, on the platform unloading, is precarious. The writers do not agree that it is the best method for handling concrete, as they, their experience being that a dinky and cars to a derrick are about 20% faster than cableway to derrick. For other general work around dam construction, it is certainly a most timely tool and one that should always be given its place in the outfit.

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